2.3.3 Horizontal Orifice

Figure 2.3.3.2 Estimated		10-year storm	1-year storm
Q	cfs	4.3	3.9
10″	S =	0.039	0.032
	V =	7.9	7.2
Top of sewer downstream		13.90	13.90
Exit loss		0.97	0.81
Friction loss		3.90	3.20
Bend loss		0.24	0.20
Subtotal		19.01	18.11
HGL above horizontal		Yes	Yes
Orifice V		(8.6)	(7.8)
Entrance loss 0.5 $V^2/2g$		0.58	0.47
Enlargement loss		0.02	0.01
HGL in orifice chamber		19.61	18.59
HGL in diversion chamber		19.60	18.50
2.0′ x 1.5 opening			
$\sqrt{}$ = Q ÷ (0.7) (3) (8.03); H =		0.07	0.05
HGL in Orifice Chamber		19.53	18.45
Difference in HGL		0.08	0.14
		Close enough	Close enough
Ratio WWF: DWF = WWF/0.5		8.6	7.8

2.4 Leaping Wiers

2.4.1 Description

Leaping weirs are of two types: (1) continuous invert type; and (2) stepped invert type.

The continuous invert type has no drop in the invert at the horizontal orifice in the bottom of the sewer to change the elevation of the invert. The stepped invert type has the upstream invert raised above the downstream invert. Since regulators usually are constructed on existing combined sewers in which no drop has been provided, this requires that a plate with a raised lip be installed on the upstream side of the opening..

Some designs provide for the installation of adjustable plates to modify the size of the opening and thus the amount of intercepted flow. However, considering the effect of bridging and clogging of the weir with debris, the necessity of making such close adjustments is questionable. It is also doubtful whether such adjustments are ever made after completion, due to the difficulty of operating nuts or bolts in a constant stream of sewage.

2.4.2. Design Guidelines

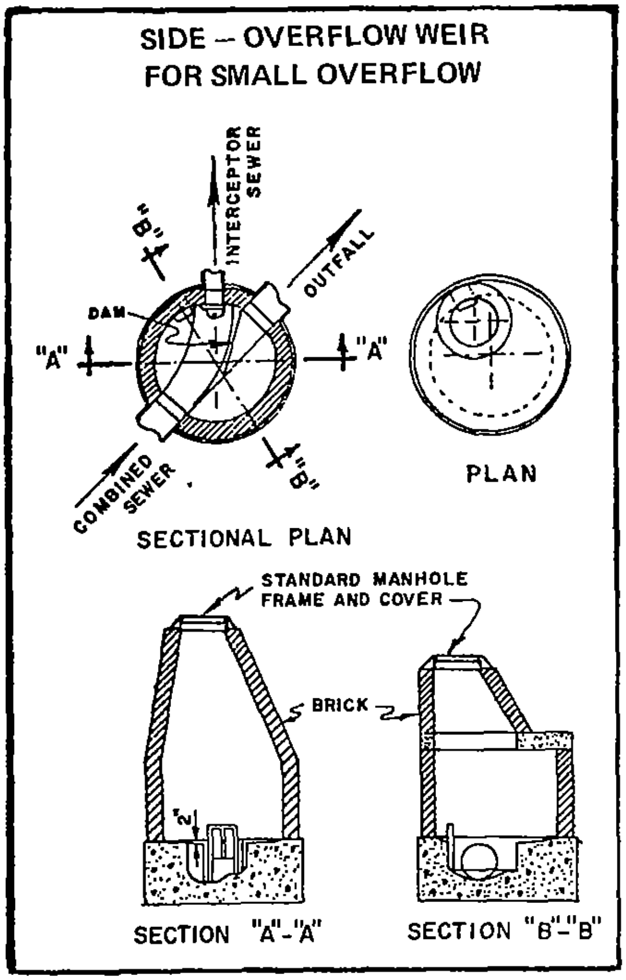
There are no generally accepted design criteria for a leaping weir. Several design methods are given in standard text books and designers are referred to these for further information.

2.5 Side-Spill Weirs

2.5.1 Description

The small regulator shown on Figure 2.5.1 illustrates how a side-spill weir can be constructed in an existing manhole. In the case shown, the designer also has added a manually operated gate at the outlet to the interceptor to further regulate the diverted flow. It should be noted that too great a restriction on the outlet may make the side-spill weir formula inapplicable.

FIGURE 2.5.1.1



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Studies recently have been carried out on the performance of side-spill weirs in England. (Ackers, P., et al., "Storm Overflow Performance Studies Using Crude Sewage.") The first spill occurred at an inflow of 0.54 cfs compared with the design figure of 0.90 cfs. The maximum flow of 0.54 cfs compared with the design figure of 0.90 cfs. The maximum flow to treatment was 1.4 cfs when the total inflow was 6.5 cfs. When the orifice was removed from the outlet pipe the flow to the interceptor was 2.6 cfs when the total flow was 8.2 cfs. The report states: "Attempts to calculate the discharges for this overflow from classic side-weir theory failed to give satisfactory agreement with observed values." It further states: "The poor degree of control achieved with the low side-weir overflow confirmed previous opinion. It spilled prematurely, as well as failing to limit flows to the desirable maximum."

A possible application of the side-spill weir using a double weir is shown in Figure 1.12.6

2.5.2 Design Guidelines

Various formulas have been presented in the past for the design of side-spill weirs. However, they have failed to be accepted and little or no data on actual performance in the field have been presented to confirm their validity. Two methods are described herein which appear to offer more reliability than previous formulas.

The characteristics of flow over side-spill weirs are related to the type of flow in the main channel. The work of investigators who dealt with this problem indicates that when the depth of flow in the main channel or pipe is at or below critical depth, with the wier height lower than critical depth or when the flow depth is greater than critical in a steep slope channel and the weir height is above critical depth, the surface curve of flow over the side-spill weir will be lower downstream. When the channel or pipe flow is at a depth greater than critical depth in a channel on a flat slope and the height of the weir crest is greater than critical depth, the surface curve over the side-spill wier will be lower at the upstream end but will rise downstream. For purposes of hydraulic analysis the assumption is made that the specific energy line H_0 referred to the channel invert remains horizontal for the length of the side-spill weir. The error due to this assumption is generally within acceptable limits.

a. Determination of Weir Discharge by Use of Q Curve

A method for determination of side-spill weir flow proposed by de Marchi[5] utilizes the concept of the Q curve which is a graphical representation of the changes that occur in a flowing stream or channel of constant cross section of flow for a fixed value of the energy level H_0.

The specific energy (referred to low point of cross section, i.e. invert of channel) is generally expressed as:

(5) Energia Elettric, July, 1941

$H_o = d + Q^2/(A^2 \times 2g)$

H_o = Energy line level

d = Depth of flow

Q = Quantity flowing

A = Area of flow

Assume the H_o value to be fixed, then

$Q = A[2g(H_o-d)]^{1/2} = 8.02A\ [H_o-d]^{1/2}$

To construct the Q curve, let d vary from zero to H_o and for each increment compute value of Q. Plot computed values of Q on horizontal axis vs. corresponding value of d on vertical axis. For a given fixed H_o the maximum value Q will be at $d_c=(H_o-d_m)^{1/2}$ where d_m is the mean depth. For a rectangular section $d_c=2/3\ H_o$; for a triangular section $d_c=4/5\ H_o$; similarly for other cross sections. The value of d_c is the critical depth. For a given energy level H_o, the maximum discharge will be at a depth d_c if the Q flowing is the necessary quantity to support that flow depth.

The following is based in part on a presentation by Prof. K. Woycicki in his book "Kanalizacke", published in 1955.

In the case of channels in which flows upstream of the side weir are at depths greater than "d_c", the determinations of diversion over the weir are started from the downstream end of the side weir. In cases where the flow depth upstream of the side weir is lower than "d_c", the determinations should begin from the upstream end of the weir. The Q curve is drawn from the channel section immediately upstream of the side-weir. The length of weir is estimated for the first calculation and then adjusted by trial. Generally, known formulas may be used for first trial. A side-spill weir flow formula adjusted by a safety factor may be utilized for trial purposes.

$Q = 2.01\,h_m{}^{3/2}$ (cfs)

(h_m = Mean value of head on weir)

For computation purposes the side weir is divided into an equal number of parts—L_1, L_2, L_3, etc. The above weir formula is also used for calculation of the partial flows. Horizontal lines are drawn to the Q curve to represent the level of the side-spill weir and the maximum flow elevation in the channel upstream or downstream as the case may be. The flow elevation downstream is determined in relation to the desired maximum flow to be delivered to treatment facilities, employing usual procedures. In the case of an upstream flow depth greater than critical, computations begin at the downstream end of the side-spill weir. The downstream water surface elevation, as above determined, minus the elevation of the weir crest, gives the weir head h_1 from which the partial flow is calculated; Subtracting the first partial discharge from Q on the Q curve establishes the next water surface level which, in turn, determines h_2 on Section l_2 Again Q_2 is computed and the procedure repeated until the entire water surface curve for the side-spill weir is established. As a check, the summation of partial Q's should equal Q_1 Q_2 the desired diversion quantity. In the case of an upstream flow depth less than critical, the procedure of calculation is similar except that computations begin at the upstream end. In the event of lack of agreement, the side-spill weir length must be modified, lengthened or shortened, and calculations repeated until agreement with $Q_1 - Q_2$ is obtained.

The above-described method entails, of necessity, a cut-and-try procedure because of the unknown varying head conditions on parts of the side-spill weir.

The theoretical considerations of the de Marchi method are predicated on the following:

1. Steady flow conditions exist;
2. Weir is in a long channel of uniform cross-sections;
3. Crest of the weir is parallel to the bed of the channel;
4. Uniform flow exists upstream and downstream of the weir;
5. Energy line is parallel to the bed of the channel; and
6. Discharge over weir may be computed by a weir-type expression.

Alternative Method for Design of Side-Spill Weirs

A more direct determination of weir length was developed by Mr. Peter Ackers in a paper published in the Proceedings of the Institution of Civil Engineers in 1957, titled "A Theoretical Consideration of Side Weirs on Storm Water Overflows."

The formulas developed by Ackers apply only to a falling profile and only when the weir height is less than half the height of the energy line relative to the channel bottom. If these conditions are satisfied the formulas presented by Mr. Ackers offer a rapid approach to the determination of required length for the side weir. This method is inapplicable with a relatively high side weir. The insertion of dip-plates (scum baffles) may greatly reduce the discharge if the clearances are small and formulas for this condition are also developed in the paper. Further this method does not take account of downstream control. Where a controlled outlet exists the method should be applied with discrimination and only where conditions are such that a falling profile would otherwise occur, i.e., c/E_w is less than 1. The paper also discusses the effect of a tapered channel on the method and states that a rate of taper in excess of the ratio of the overflow per unit length to the channel the weir is set relatively low.

The paper develops a general differential equation for the water profile along a side weir and by substitution of certain factors derives formulas for the length of weir based on a selected ratio of upstream head on the weir to downstream head on weir. The theoretical profile is shown in Figure 2.5.2. The formulas are as follows:

Ratio n	Formula for L
5	$L = 2.03B\,(2.81 - 1.55\, c/E_W)$
7	$L = 2.03B\,(3.90 - 2.03\, c/E_W)$
10	$L = 2.03B\,(5.28 - 2.63\, c/E_W)$
15	$L = 2.03B\,(7.23 - 3.45\, c/E_W)$
20	$L = 2.03B\,(8.87 - 4.13\, c/E_W)$

Notes to above: Add 10% for broad-crested weir. Halve length for double-sided weir.

The notation used is as follows:

L = length of weir

h_1 = upstream head on weir

h_2 = downstream head on weir

$n = h_1/h_2$

c = height of weir crest above invert

B = width of channel or dia. of pipe

E = specific energy related to invert

E_W = specific energy related to weir crest

α (alpha) = velocity correction coefficient

β (beta) = pressure correction coefficient.

Upstream of the weir α is 1.2 and β is unity.

Along weir α is 1.4 and β is 0.8

The original paper gave the following formula for computing total head based on flow upstream of the weir.

$$E_W = 1.2\, V^2/2g + (d_n - c) \qquad \text{(Equation 1)}$$

However, in discussion published subsequently Ackers stated the use of this equation "could lead to anomalies." Therefore, he suggests computing the head at the upstream end of weir based on the assumption that $h_1 = \frac{1}{2} E_W$ which results in following:

$$E_W = \alpha Q^2/2_g A^2{}_1 + \tfrac{1}{2}\beta E_W \qquad \text{(Equation 2)}$$

If the area of the water in a rectangular channel, i.e. $B\,(c + \frac{1}{2} E_W)$ is substituted for A_1 equation 2 becomes:

$$[E_W/c] \times [1 + (E_W/2c)]^2 = \alpha/(2-\beta) \times Q^2/(gB^2C^3)$$

(Equation 3)

Figure 2.5.3 b was developed to solve this equation for c/E_W for rectangular channels.

For circular sections the procedure might be the use of equation 1 for a preliminary value of E_W and then the use of equation 2 by trial and error for a more exact value.

Having determined c/E_W and having selected the ratio of n, the required length of weir can be determined by use of Figure 2.5.2, or by the equations given previously:

Example by Ackers Method:

A combined sewer 48 inches in diameter constructed on a slope of 0.002, with Manning n of 0.013 has a capacity of 65 cfs flowing full. The average dry-weather flow is 6.5 cfs. It is desired to divert flows in excess of 2 x DWF

Upstream flow = 65 cfs

Upstream sewer = 48 in. dia.

Flow to be diverted = 52 cfs

d_c = critical depth

d_n = normal depth

D = diameter

Q = sewer capacity

q = design flow

(1) Critical flow depth in 48-inch-diameter pipe vs. maximum flow depth: From Figure 26, ASCE manual No. 37

$d_c/D = 0.60$, $d_c = 0.60 \times 4.0 = 2.4$ feet

$d_n = 0.8 \times 4.0 = 3.2$ feet

Since d_n is greater than d_c, drawdown will occur at side weir:

(2) Determine c

for Q = 2 x DWF = 13 cfs

$q/Q = 13/65 = 0.20$

$d/D = 0.30$, $d = 0.30 \times 4.0 = 1.20$ feet

therefore $c = 1.20$

Weir crest must be set 1.20 feet above invert so that 2 x DWF can continue directly downstream.

(3) Determine c/E_W

Assume Figure 2.5.3 B is applicable for circular channels.

$C/[Q^2/gB^2]^{1/3} = 0.59$

From Figure 2.5.2 B $c/E_W = 0.60$

Since c/E_W is less than 1.0 a falling profile will develop.

(4) Determine L

Use $n = 5$ and $c/E_W = 0.60$

From Figure 2.5.2 c $L/B = 3.9$

$L = 3.9 \times 4.0 = 15.6$ feet or

7.8 feet per side if double weir is used.

(5) Determine h_1 and h_2

$c/E_W = 0.60$

$E_W = 1.20/0.60 = 2.00 = 1.00$ feet (double weir)

$h_1 = 0.5\, E_W = 1.00$ feet

$h_2 = 1.00/5 = 0.20$ feet

(6) Determine flow to plant

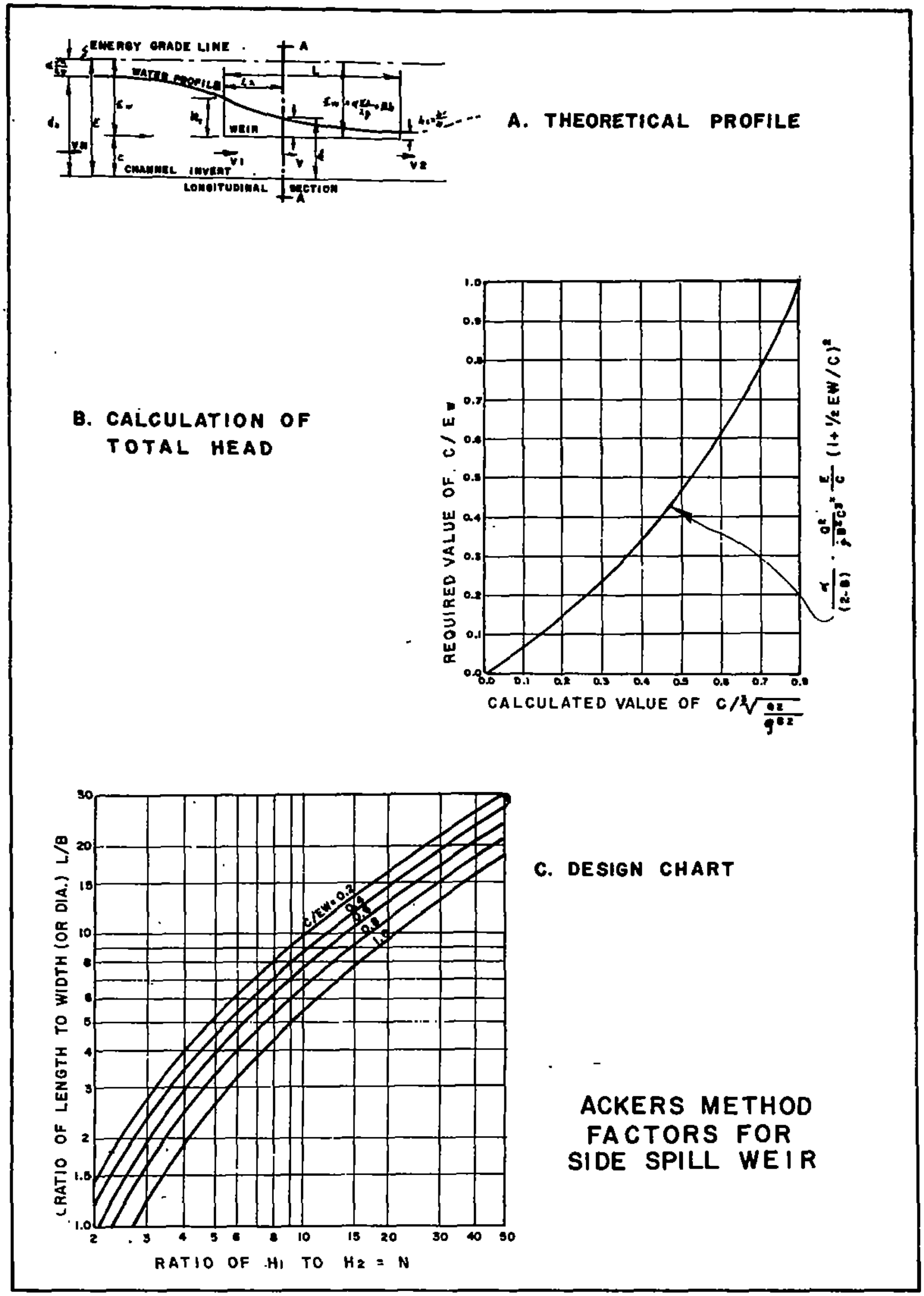
$d = c + h_2 = 1.20 + 0.20 = 1.40$

$d/D = 1.40/4.00 = 0.35$

$q/Q = 0.26$ (from chart of hydraulic properties)

$q = 0.26 \times 65 = 17$ cfs which is greater than 13 cfs desired

FIGURE 2.5.2



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(7) Determine flow to plant for n = 10

If n = 10 were selected then

$L = 7.5 \times 4.0 = 30$ feet

$h_2 = 1.00/10 = 0.10$ feet and

$q = 15$ cfs

2.6 Internal Self-Priming Siphons

2.6.1 Description

Self priming siphons may be classified into two types: (1) Internal self-priming where the siphon action is induced by flow in the siphon, and (2) external self-priming where the siphon action is induced by flow in a priming tube situated outside the siphon. An internal type is shown in Figure 2.6.1. The following discussion relates to internal self-priming siphons.

The internal self-priming siphon consists of: (1) Entrance section; (2) upflow leg; (3) vertical throat section; (4) downflow leg; and (5) outlet section. The downflow leg is designed with an adverse slope to aid in creating a negative pressure at the summit. As the water level rises to the crest of the siphon the water flows over the crest in a sheet and strikes the opposite wall of the downflow leg thus sealing the siphon. As the sheet falls it carries air from the summit with it. When the upstream water level rises enough to seal the air vent the falling sheet of water carries out the remaining air in the summit and the siphon discharges at full capacity. The siphon continues to discharge until the upstream water surface falls below the air vent and enough air is admitted to the siphon summit to stop the siphonic action. For quick priming action it is advisable to provide a water seal at the siphon outlet.

2.6.2 Design Guidelines

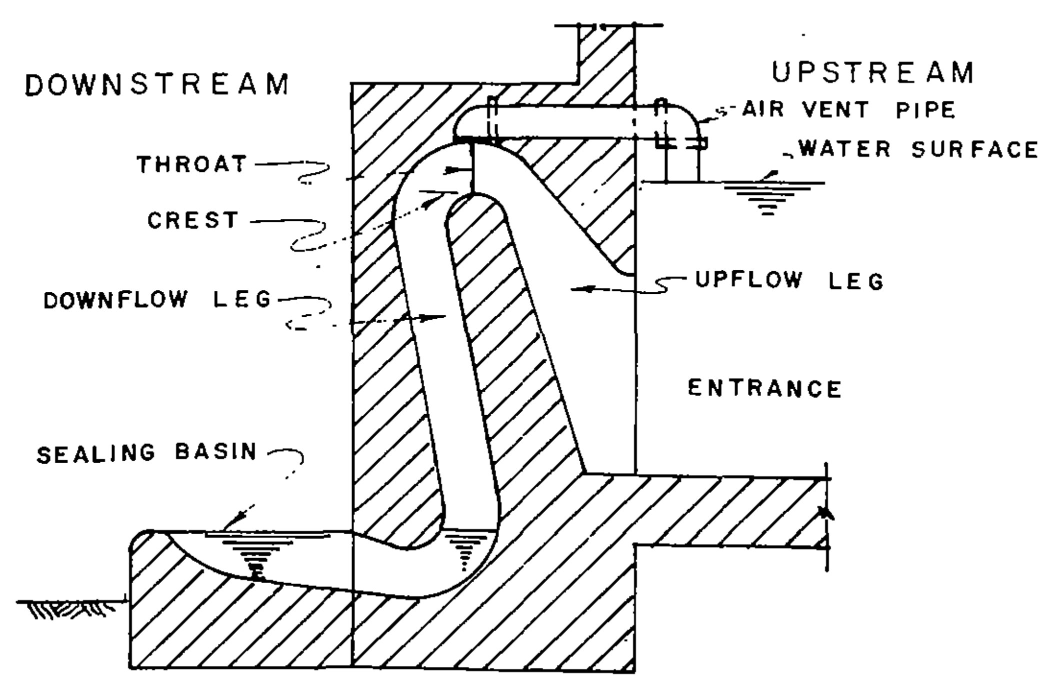
The use of this type of siphon is considered herein for discharging excess storm flows to the receiving waters.

The energy equation for flow through a siphon is:

$H = V^2/2g + K\,V^2/2g + f\,(l + V^2)/(D + 2g)$

where H = difference in elevation of upstream and downstream water surface in feet

FIGURE 2.6.1



INTERNAL SELF-PRIMING SIPHON

v = velocity in fps

f = friction coefficient

l = length of siphon in feet

D = diameter of siphon in feet

K = loss coefficients for entrance, transition, bend and exit losses.

Due to the difficulty in determining the proper loss coefficients in the above equation the following equation also has been used:

$Q = CA (2 gH)^{1/2}$ where

Q = discharge in cfs

C = coefficient

A = area of throat in sq. ft.

H = same as above in ft.

The value of C may vary from 0.3 to 1.0 but generally will range from 0.5 to 0.85 in a well-designed siphon.

Design criteria for siphon and values of the various loss coefficients are given in paragraph 207 of "Design of Small Dams, Bureau of Reclamation, First Edition 1960." Figure 237 of that publication is a chart for determining the value of C for use in an equation similar to the one given above.

Due to the uncertainty in selecting the proper C value, some engineers in the past have recommended this be determined by model test of proposed siphons.

On a recent project in England, in 1958, siphons were made of sheet copper and tested before installation. The copper siphons were then encased in concrete during construction.

The following criteria are based on British experience:

1. The air vent pipe should have a minimum area of six percent of the throat area.
2. A sealing basin at the outlet is necessary for efficient priming.
3. A two-inch depth of flow over the crest at the summit is the maximum necessary to prime the siphon.

DYNAMIC REGULATORS – SEMI-AUTOMATIC

2.7 Float Operated Gates

2.7.1 Description

This regulator may consist of three chambers: (1) Diversion chamber; (2) regulator chamber; and (3) tide gate chamber, when required (Figure 2.7.1).

The diversion chamber contains a dam to deflect the dry-weather flow into the regulator chamber. In flat regions, the diversion dam usually is set a maximum height of six inches above the invert of the combined sewer to minimize backwater effects upstream in the combined sewer during storm flows. The diversion channel invert is established so that peak dry-weather flow can be diverted without flowing over the dam. Excess storm flow will pass over the dam into the tide gate chamber and to overflow.

The regulator chamber contains the float, the regulating gate and the interconnecting linkage between the float and the gate. The gate is installed on an opening in the common wall between the regulator and diversion chambers. Usually the float is situated in a well which is connected by a telltale passage to the channel of the combined sewer in the diversion chamber, or to the channel of the intercepted flow in the regulator chamber.

There are three principal methods of controlling the regulating gate. These are designated herein as Types A, B, and C. In the Type A control the telltale passage extends from the float well to the combined sewer and thus reflects the water level in the combined sewer. This method is used if it is desired to prevent any diversion of flow to the interceptor when the water surface in the combined sewer reaches a certain level.

In the Type B control the telltale passage extends from the float well to the flow channel in the regulator chamber. This type of control is used if it is desired to divert a certain quantity to the branch interceptor before reducing the amount diverted from the combined sewer.

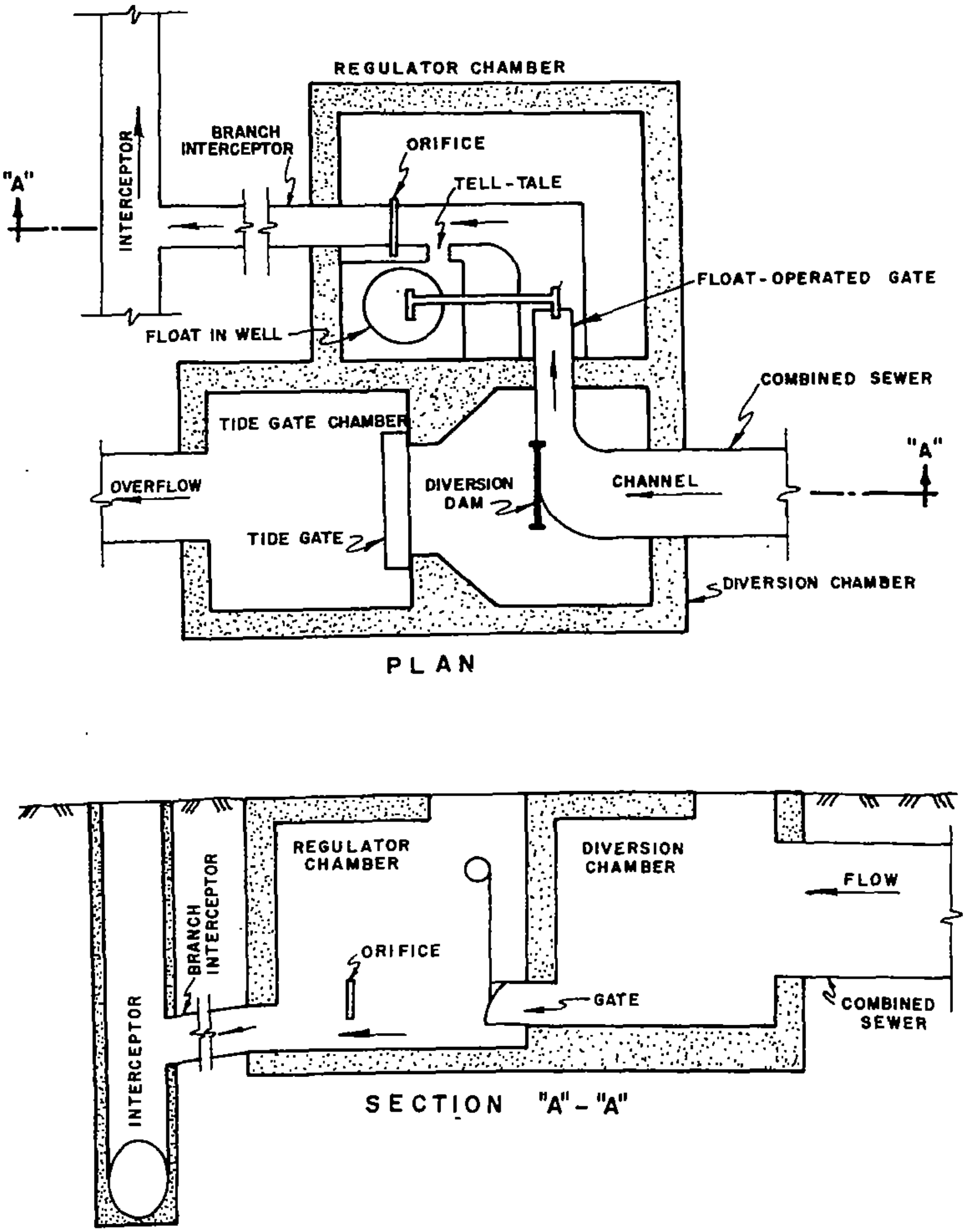
The Type C control is similar to Type B except that an orifice plate is installed in the flow channel downstream from the entrance of the telltale to the channel. This type of control is used if it is desired to pass a predetermined quantity through the regulator regardless of conditions in either the combined sewer or the interceptor. This type of control generally can be designed so that the desired discharge can be controlled within plus or minus 5 to 10 percent. Type C control is considered the most desirable and is used herein for illustrative purposes. A typical layout of a regulator using the Type C control is shown on Figure 2.7.1.

2.7.2 Design Guidelines and Formulas

Type C Control

For design, obtain all pertinent data for the combined sewer at the proposed location of the regulator including diameter, invert elevation, slope, average and peak dry weather flow and peak storm flow. Similar data must be obtained for the interceptor at the proposed junction with the branch interceptor. If the interceptor is being designed in

FIGURE 2.7.1



TYPICAL LAYOUT
FLOAT-OPERATED GATE

conjunction with the regulator, assume the elevation for the interceptor and adjust as necessary by subsequent computations.

First, set the limits on the maximum flow to be diverted to the interceptor. Usually the object is to divert the peak dry-weather flow to the interceptor and hence this value is selected as the maximum dry-weather discharge through the regulator. This represents the maximum discharge with the gate fully open and only dry-weather flow in the combined sewer. Then the maximum discharge through the regulator in wet weather will be the peak dry-weather flow plus a minimum amount varying 10 to 20 percent of the peak dry-weather flow—the maximum discharge through the partially closed gate with storm flow in the combined sewer. This minimum variation between the maximum dry-weather diversion and the maximum wet-weather diversion is necessary to provide adequate variation in the water surface in the float well to cause sufficient float travel as explained hereafter.

For dry weather conditions the total available head loss is divided between h ft., the head loss through the gate and H ft., the head loss through the orifice. Likewise, under maximum storm conditions the total available head is split between h inches, the head loss through the gate and H inches, the head loss through the orifice. The difference in the water surface upstream of the orifice under dry-weather and storm conditions determines the float travel. The difference in the water surface in the combined sewer under dry-weather and storm conditions determines the amount the gate must close, or the shutter travel. If the ratio of shutter travel to float travel exceeds 2 then: (1) A new head loss must be chosen for the orifice; or (2) the discharge through the regulating gate during storms must be increased.

Computations may be made in the following steps:

1. Design branch interceptor for peak dry-weather flow.
2. Using peak dry-weather flow, determine hydraulic gradient at the exit of the regulator chamber on the following basis:
   a. Determine the water surface in the interceptor. If a drop manhole is required at the interceptor so that flow at critical depth occurs, then investigate to see if the branch interceptor is long enough for the backwater curve to attain normal depth at the upstream end of the branch interceptor. Compute the hydraulic profile upstream to the regulator chamber.
   b. Determine the water surface in the regulator chamber. Select channel width. Determine critical depth. Make flow depth 15 percent or more greater than the critical depth. Determine the energy line, hydraulic grade line and invert.
3. Determine total available head between the hydraulic grade line at the exit of the regulator and the dam elevation in the combined sewer. To minimize backwater effect in the combined sewer during storm flows, it may be advisable to set the dam a maximum of six inches above the invert of the combined sewer.
4. Using about one-half of the total available head as H ft., determine the orifice area.
Use $Q = CA\,(2gH_2)^{1/2}$ with $C = 0.70$.
5. Determine the hydraulic grade line immediately downstream of the orifice. Use $d_s = d_2\ [1+(2V_2{}^2/gd_2) \times (1-d_2/d_1)]^{1/2}$ (King 5th, Equation 4-25)
Where d_s = depth of flow downstream of orifice
   d_2 = depth of flow downstream below turbulence
   V_2 = velocity at d_2
   d_1 = height of orifice
   Q = quantity
   A = area
   C = coefficient
6. Determine the total available head on the basis of d_2 above.
7. Determine h ft. (head loss through gate).
8. Select regulating gate.
Use $Q = CA\ (2gH)^{1/2}$ with $C = 0.95$ to obtain required area. Select the nearest regulating gate size. On the basis of the size selected, determine h ft. (Head loss through gate.)
9. Check orifice size. Determine new H ft. on the basis of h ft. determined in Step 8. Repeat computation for total available head and orifice size until the error is minor.
10. Establish regulator gate elevations so that the gate is submerged on the downstream side with peak dry-weather flow.

Using the maximum wet-weather diverted flow, proceed as follows:

11. Determine the hydraulic grade line at the regulator chamber exit. Proceed upstream from the upstream end of branch interceptor, selecting trial depths and comparing the energy line in the channel with the sum of the energy line in the branch interceptor and entrance loss.
12. Determine H in., the head loss through the orifice. Use $Q = CA\ (2gH)^{1/2}$ with $C = 0.7$.
13. Check the hydraulic grade line downstream

of the orifice, using equation in Step 5.

14. Determine the hydraulic grade line upstream of the orifice.

15. Determine the float travel (F.T.) as the difference in the hydraulic grade line upstream of orifice for peak dry-weather flow and the maximum storm diverted flow.

16. Determine shutter travel (S.T.), the amount the shutter of the gate must close so that the discharge during storm flow will not exceed the maximum diverted flow. It is necessary to use manufacturer's charts for this computation since the coefficient of discharge for the gate varies with the closing of the gate.

17. Determine ratio of S.T. to F.T. If this ratio is less than 2, design is satisfactory. If ratio is greater than two, then redesign must be made, as stated above.

Sample computations follow. The hydraulic profiles for these computations are shown in Figure 2.7.3.

In the design computations friction head losses in the channels are neglected since these losses are minor. It is also assumed that there is complete loss of velocity head of the flow entering the diversion chamber and of the flow entering the regulator chamber. A 90-degree bend in the channel of the regulator chamber is considered advisable to eliminate the latter velocity head so that the orifice design may disregard the effect of approach velocity.

2.7.3 Sample Computation
Float-Operated Gate

<u>Pertinent data</u>

Interceptor sewer

D = 60", Invert el. = 10.0 ft., W.S. = 14.0 ft. for 34.2 cfs diversion and 13.77 ft. for 30 cfs diversion

Combined sewer

D = 60", Invert el. = 17.0 ft., S = 0.0022
Manning n = 0.013, V (full) = 6.2 fps

	Q	cfs	d ft.	W.S. El.	V fps
DWF =	Av. dry-weather	15	1.20	18.20	4.3
	Peak dry-weather	30	1.70	18.70	5.2
	Peak storm	120	4.00	21.00	7.0

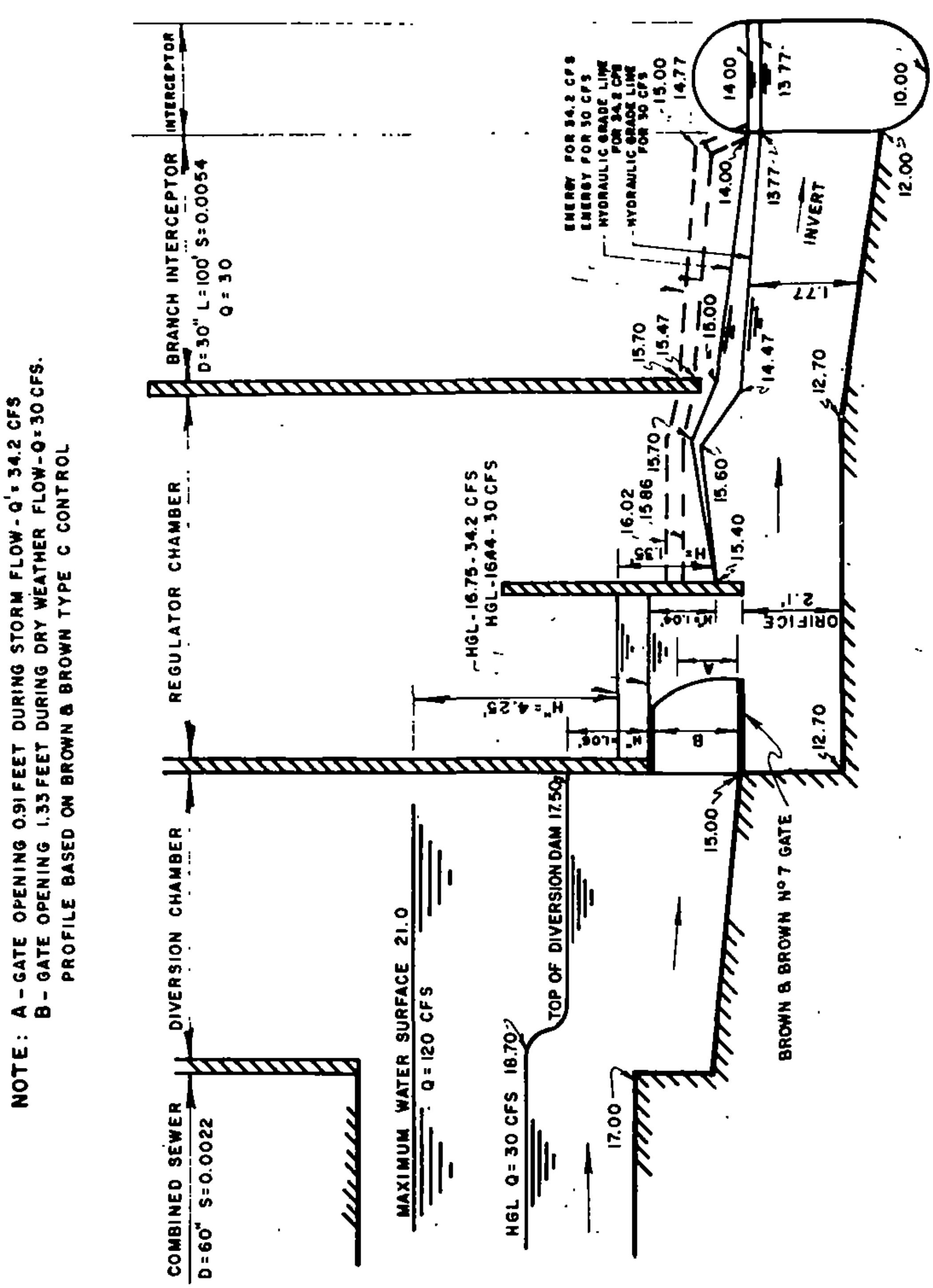
Distance from interceptor to regulator on combined sewer is 100 ft.

HGL = hydraulic grade line
EL = energy line

Use Brown & Brown Regulator with Type C Control. Maximum diversion will exceed design diversion by 10 to 20%. Use 14%.

h' = head loss thru regulator gate during dry-weather flow

FIGURE 2.7.3



HYDRAULIC PROFILES FLOAT-OPERATED GATE

2.7.3 Float-Operated Gate

h″ = head loss thru regulator gate during storm flow

H′ = head loss thru orifice during dry-weather flow

H″ = head loss thru orifice during storm flow

d = depth of flow

D = diameter

d_c = critical depth

	ELEVATION		
	Invert	HGL	EL
Regulator should pass peak dry-weather flow			
Design diversion = Q = 30 cfs			
Maximum diversion = Q′			
Q + 14% = (30) (1.14) = 34.2 cfs = Q′			
Interceptor	10.00	14.00 13.77	
Branch Interceptor Use n = 0.013			
Determine hydraulic profile			
Q′ = 34.2 cfs			
Use D = 30 in., S = 0.007			
V (full) = 7.1 fps			
V = 1.13 x 7.1 = 8.0 fps @ 2′ flow depth			
$V^2/2g = 1.00$			
d = 0.8 x 2.5 = 2.0			
downstream end	12.00	14.00	15.00
Rise = 100 x 0.007 = 0.70			
upstream end	12.70	14.70	15.70

Q = 30.0 cfs
$Q/Q^1 = 0.88$
d = 0.71 x 2.5 = 1.77 ft.
V = 1.12 x 7.1 = 8.0

2.7.3 Float-Operated Gate

	ELEVATION		
	Invert	HGL	EL
$V^2/2g = 1.00$			
downstream end	12.00	13.77	14.77
Rise = 0.70			
upstream end	12.70	14.47	15.47

Regulator Chamber Q = 30 cfs
Use channel width = 2.5 ft.

$d_c = 1.62$ (Fig. 37 ASCE Manual 37)

For stable flow d = 1.5 x 1.61 = 1.86

Determine d at chamber exit by trial for energy balance

	Invert	HGL	EL
d = 2.9 ft. HGL = 12.70 + 2.9	12.70	15.60	
V = 30 ÷ (2.9) (2.5) = 4.13 fps			
$V^2/2g = 0.26$ EL′ = 15.60 + 0.26			15.86

Entrance loss = 0.5 (1 – 0.26) = 0.37

EL = 15.47 + 0.37 = 15.84 < 15.86

Check OK

Neglect friction head loss in channel

Determine total head

Diversion dam = 17.00 + 0.50 =	17.50
W.S. regulator	15.60
Total head—1st trial	1.90

Trial H′ = 1.90 ÷ 2 = 0.95

Determine orifice size

$Q = C A \sqrt{2 gH}$

$30 = (0.70) A (8.03) \sqrt{0.95}$

$A = 5.48$

Try orifice = 2.5 ft. wide by 2.19 ft. high

2.7.3 Float-Operated Gate

	ELEVATION	
Invert	HGL	EL

Determine HGL downstream of orifice

$$d_S = d_2 \left[1 + \frac{2V_2^2}{gd_2} \left(1 - \frac{d_2}{d_1}\right) \right]^{1/2}$$

(from King 5th, equation 4–25)

$$= 2.9 \left[1 + \frac{(2)\,(4.13)^2}{(32.2)(2.9)} \left(1 - \frac{2.9}{2.19}\right) \right]^{1/2}$$

$$= 2.72$$

HGL = 12.70 + 2.72 = 15.42 — Invert 12.70, HGL 15.42 Trial

Determine total head

Diversion dam	=	17.50
HGL downstream orifice		15.42
Total head - 2nd Trial		2.08

Determine regulating gate size

Trial h' = total head - H'

= 2.08 – 0.95 = 1.13

$Q = CA\sqrt{2gH}$

$30 = (0.95)\,(A)\,(8.03)\sqrt{1.13}$, $A = 3.70$ sq. ft.

From Brown & Brown Catalog

Use Gate No. 7 A = 3.81

16 in. high x 34¼" wide

Determine h' based on Gate No. 7

$30 = (0.95)\,(3.81)\,(8.03)\sqrt{h'}$

Final h' = 1.06

Re-determine orifice size

H' = 2.08 – 1.06 = 1.02 ft.

$30 = (0.7)\,(A)\,(8.03)\sqrt{1.02}$

A = 5.30 sq. ft.

Orifice = 2.5 ft. wide x 2.12 ft. high

Re-check Total head

$D_S = 2.70$

			Invert	HGL
HGL downstream orifice	=	15.40	12.70	15.40
Diversion dam	=	17.50		Final
Final total head	=	2.10		

2.7.3 Float-Operated Gate

	ELEVATION Invert	HGL	EL
Re-determine orifice size			
Final $H' = 2.10 - 1.06 = 1.04$			
$A = 5.25$ sq. ft.			
Orifice $= 2.5$ ft. wide			
2.1 ft. high			
Upstream of orifice			
$HGL = 15.40 + H'$			
$= 15.40 + 1.04$	12.70	16.44	16.44
Regulator Chamber			
Determine setting of regulating gate			
Submerge by 0.11 ft.			
Top = $16.44 - 0.11 = 16.33$			
Invert = $16.33 - 1.33 = 15.00$			
Determine conditions for $Q' = 34.2$ cfs			
Head loss at orifice = H''			
$Q' = CA\sqrt{2gH}$			
$34.2 = (0.7)\,(5.25)\,(8.03)\sqrt{H''}$			
$H'' = 1.35$ ft.			
Upstream end of branch interceptor	12.70	14.70	15.70
Determine d at chamber exit by trial for energy balance			
$d = 3.0 + HGL = 12.70 + 3.0$	12.70	15.70	
$V = 34.2 \div (3.0)\,(2.5) = 4.56$			
$V^2/29 = 0.32$ EL $- 15.70 + 0.32$			16.02
Entrance loss = $0.5\,(1.00 - 0.32) = 0.32$			
EL = $15.70 + 0.34 = 16.04 > 16.02$			
Check OK			

Check HGL downstream of orifice

$$ds = d_2 \left[1 + \frac{2v_2^2}{gd_2} \left(1 - \frac{d_2}{d_1} \right) \right]^{1/2}$$

$$= 3.0 \left[1 + \frac{(2)\,(4.56)^2}{(32.2)(3.0)} \left(1 - \frac{3.0}{2.1}\right) \right]^{1/2}$$

$$= 2.70$$

2.7.3 Float-Operated Gate

	ELEVATION		
	Invert	HGL	EL
HGL = 12.70 + 2.70 = 15.40		15.40	
HGL upstream of orifice			
HGL = 15.40 + H″ = 15.40 + 1.35		16.75	

Regulator Chamber

Float travel = F.T.

HGL for 34.2 cfs = 16.75
HGL for 30.0 cfs = 16.44
F.T. = 0.31

h″ = 21.00 – 16.75 = 4.25 ft.

From Brown & Brown Catalog

C = coefficient of discharge
P = % of gate opening

$$CP = \frac{Q'}{A\sqrt{2gh''}} = \frac{34.2}{(3.81)(8.03)\sqrt{4.25}}$$
$$= 0.54$$

From chart D.S. 347, Brown & Brown

For CP = 0.54
C = 0.79
and P = 68.5%

Height of gate opening = .685 x 1.33
= 0.91 ft.

S.T. = gate shutter travel
S.T. = 1.33 – 0.91 – 0.42 ft.

$$\frac{S.T.}{F.T.} = \frac{0.42}{0.31} = \frac{1.35}{1} < \frac{2}{1}$$

2.8 Tipping Gates

2.8.1 Description

The regulator structure is similar to that required for manually operated gates. A typical regulator with tipping gate and flap gate is shown in Figure 2.8.1.1.

The detail of the tipping gate as used in Milwaukee about 1919 is shown in Figure 2.8.1.2. The horizontal pivot is located so that one-third of the plate is below the pivot. The housing for the gate is precast concrete and was made with opening widths of 12, 18, and 24 inches.

The detail of the gate as used recently is shown in Figure 2.8.1.3. This gate is made in opening widths of 8, 12, 24, and 36 inches. Where an opening wider than 36 inches is required, multiple gates are used. This model of tipping gate is used by the Allegheny County Sanitary Authority, Pittsburgh, Pa. The gate differs from the gate used in Milwaukee in that: (1) The housing is of cast metal rather than concrete; (2) the bottom third of the plate below the pivot has a deflection angle of about 24 degrees with the upper part of the plate; (3) the top of the housing is curved to provide minimum clearance between the housing and the top of the plate as the latter rotates; (4) the maximum and minimum opening can be adjusted; and (5) the bottom is fixed rather than adjustable.

2.8.2 Design Guidelines

For design, obtain all pertinent data for the combined sewer at the proposed location of the regulator, including diameter, invert elevations, slope, average and peak dry-weather flow and peak wet-weather flow. The design flow is considered herein to be the peak dry-weather flow. Similar data for the interceptor at the proposed junction with the collector are required.

Compute the hydraulic grade and energy lines for peak dry-weather flow, starting at the interceptor and proceeding upstream along the branch interceptor to the tipping gate chamber. Determine the elevation of the diversion dam in the diversion chamber. Determine the differential head available and select the size of gate and gate opening from Figure 2.8.2. If the available differential head is greater than that required for the design flow, the head can be decreased by raising the branch interceptor and raising the water surface downstream of the gate. Since the data for Fig. 2.8.2 are based on a minimum downstream water depth of 10 inches, the tipping gate should be set with its invert 10 inches below the downstream water surface. If the differential head is not large enough to provide discharge equal to the design flow, the branch interceptor should be redesigned, or a larger gate used.

After selecting the gate opening required to pass the peak dry-weather flow, the minimum gate opening during wet-weather periods should be determined. This is done by the trial and error method using the following steps: (1) Assume diverted discharge; (2) determine downstream water surface; (3) determine differential head; (4) select opening from Fig. 2.8.2 for differential head nearest the assumed discharge; and (5) repeat until the assumed discharge is close to the discharge selected from Fig. 2.8.2 and approximates the design diverted flow.

Sample computations are given in paragraph 2.8.4. The given conditions are the same as those used in the sample computations for the manually operated gate. It should be noted that the ratio of peak wet-weather flow (WWF) to average dry-weather flow (DWF) is 4.9 for the tipping gate, compared to 6.8 for the manually operated gate. For this reason the tipping gate is considered preferable to the manually operated gate.

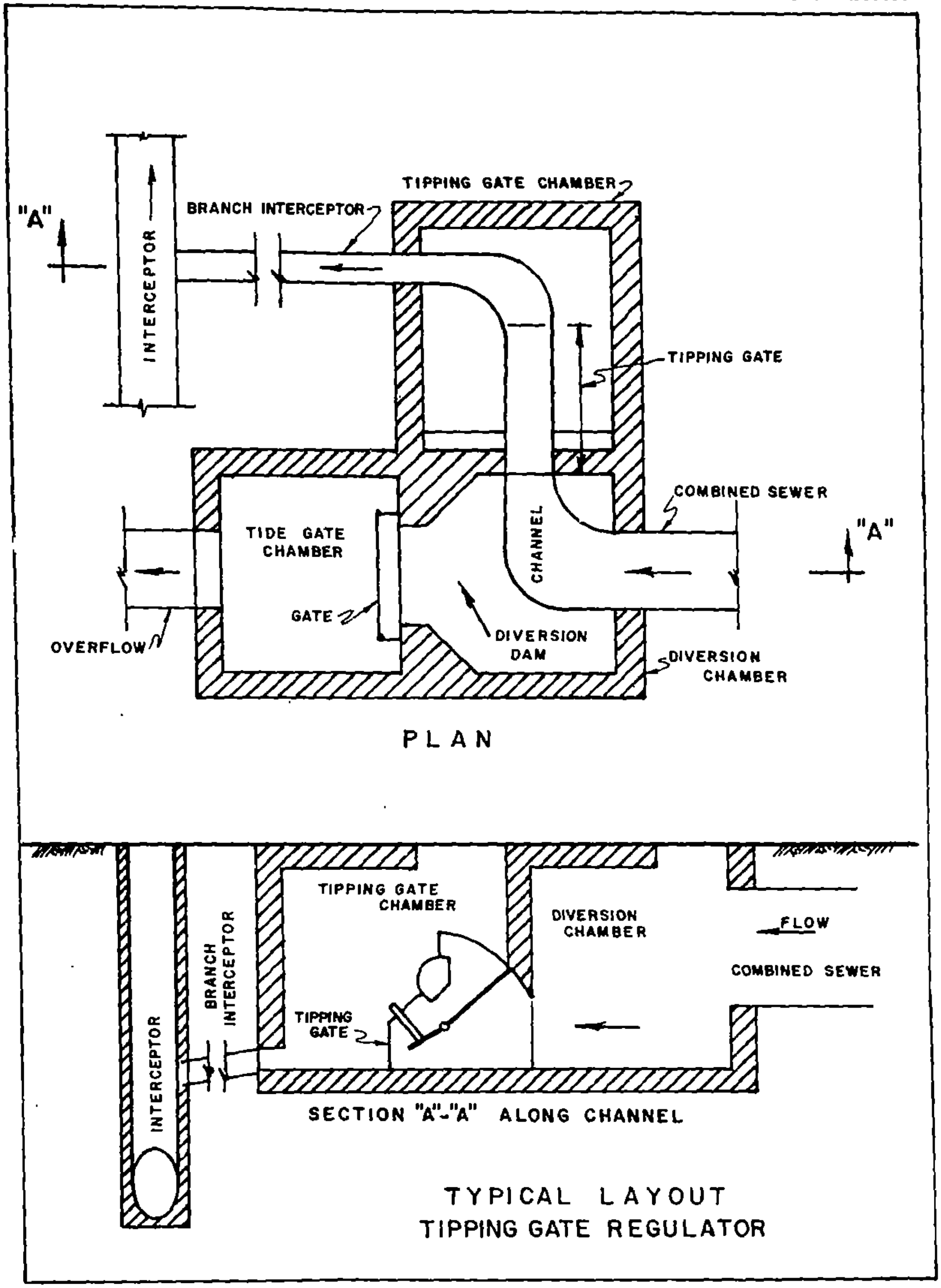
The hydraulic profile for the design diverted flow of 2 cfs is given in Fig. 2.8.4.2. The hydraulic profile for the maximum diversion of 2.45 cfs in storm periods is shown in Fig. 2.8.4.3. Fig. 2.8.4.1 illustrates the hydraulic conditions affecting the gate. At the diversion equal to the peak dry-weather flow of 2 cfs, the differential head (A) is 0.4 feet and the gate opening is 5.0 inches. When the differential head (C) is 0.9 feet, the upstream head is 2.0 feet and the gate begins to close. When the flow reaches its maximum elevation of 19.6 feet in the combined sewer the differential head (B) is 2.89 feet, the opening in the gate has been reduced to 2.56 inches and the flow diverted to the interceptor through the gate is 2.45 cfs.

2.8.3 Design Formulas

In connection with their contract for furnishing tipping gate regulators to the Wyoming Valley Sewerage Project, the Rodney Hunt Company was required by Albright and Friel, the project engineers, to have the gate calibrated by laboratory test. These tests were made at the Alden Research Laboratories, Worcester Polytechnic Institute, Worcester, Mass. The following charts from this report are reproduced through the courtesy of the *Wyoming Valley Sewerage Authority.*

a. Figure 2.8.2 shows the relation between discharge through a 12-inch wide gate and the differential head between the water levels upstream and downstream level constant at 10, 16, 22, and 28 inches and varying the upstream head. Where two curves are shown for a given opening the lower curve represents the flow for a downstream elevation of 10 inches. Otherwise

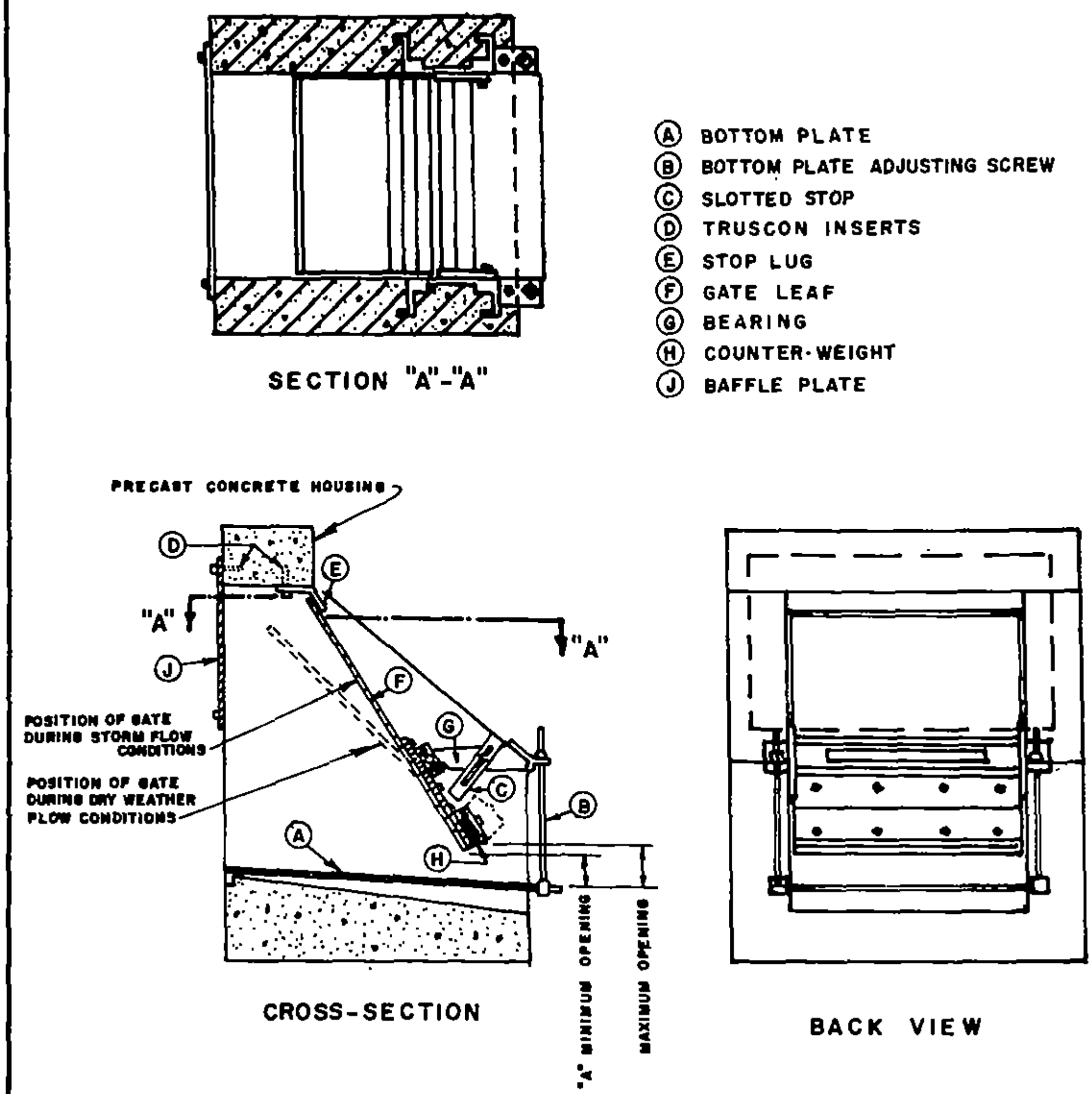
FIGURE 2.8.1.1



TYPICAL LAYOUT
TIPPING GATE REGULATOR

Courtesy Rodney Hunt Co.

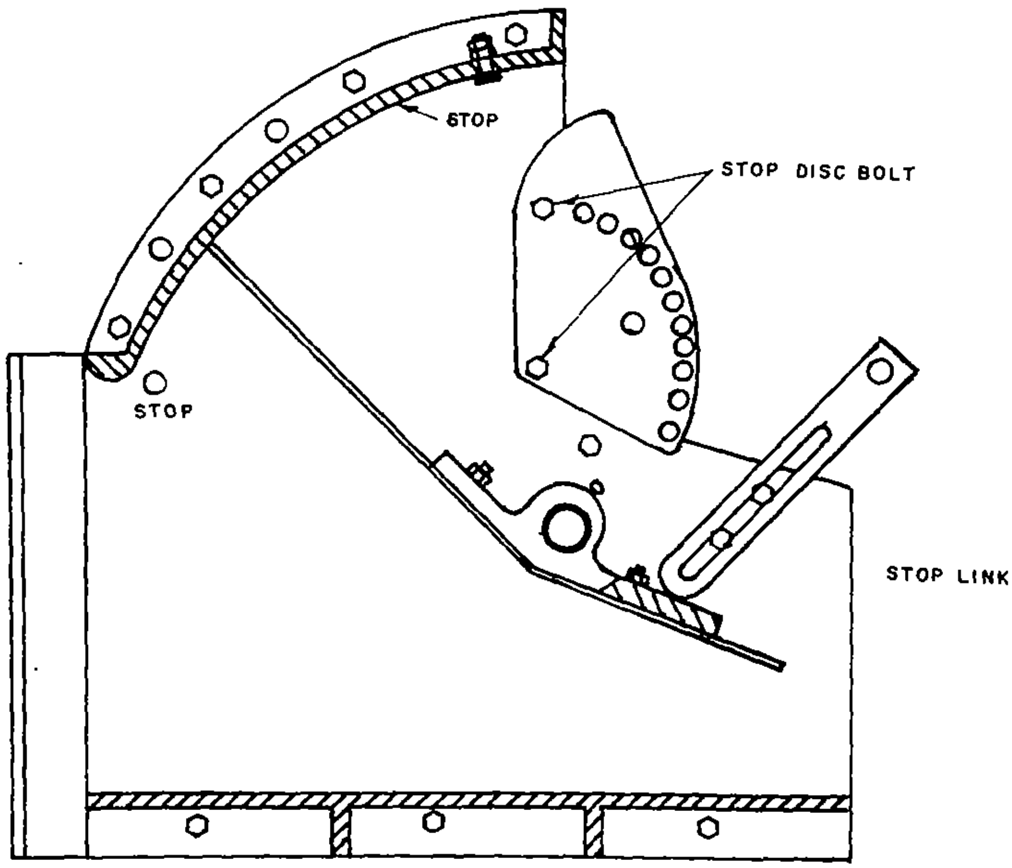
FIGURE 2.8.1.2



TIPPING GATE
USED AT MILWAUKEE

Courtesy Rodney Hunt Co.

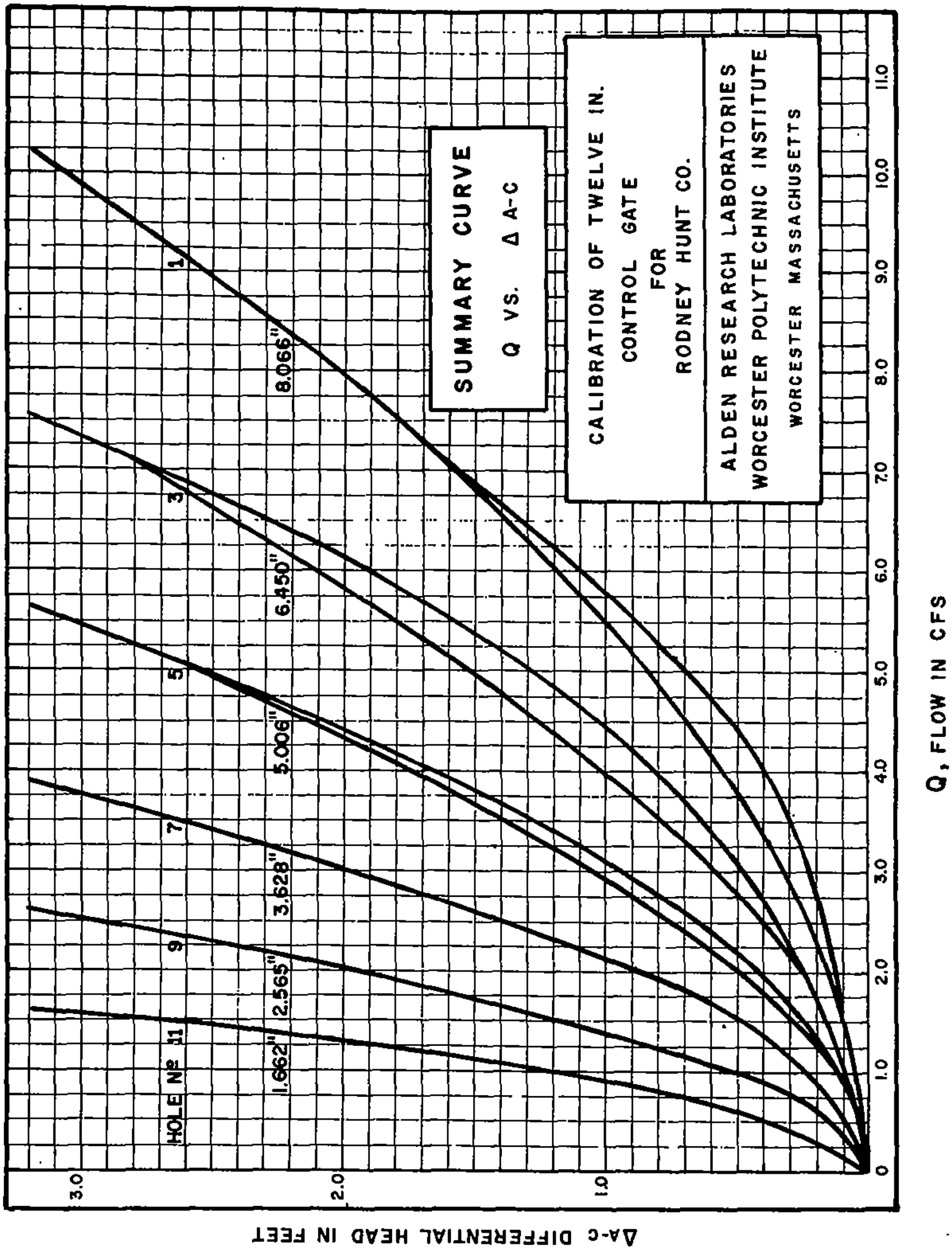
FIGURE 2.8.1.3



TIPPING GATE
USED BY ALLEGHENY COUNTY
SEWAGE AUTHORITY

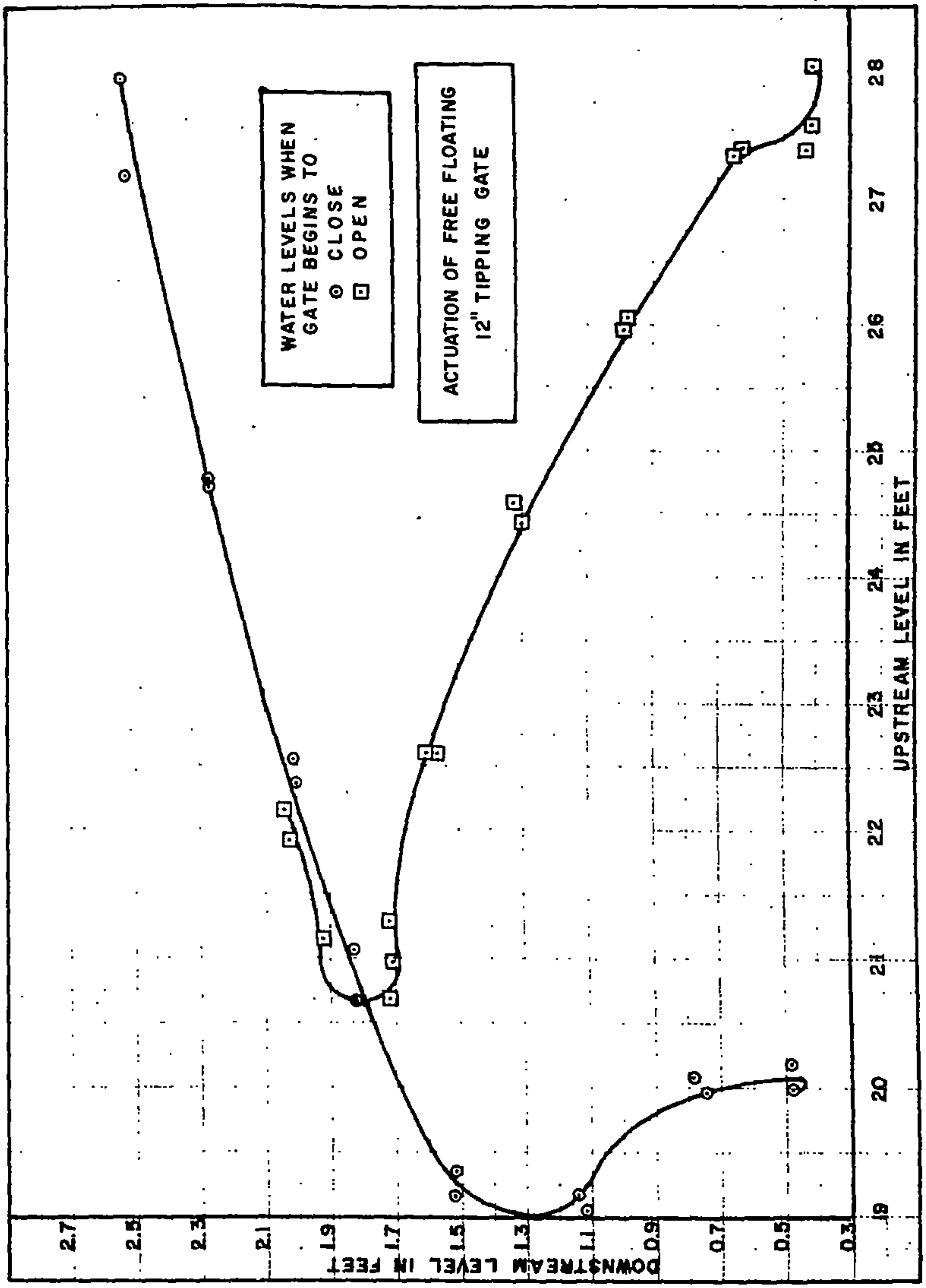
Courtesy Rodney Hunt Co.

FIGURE 2.8.2



Courtesy Rodney Hunt Co. and Alden Research Laboratories

FIGURE 2.8.3



Courtesy Rodney Hunt Co. and Alden Research Laboratories the curves represent the average of the data for all downstream heads.

b. Figure 2.8.3 shows upstream water levels at which the gate starts to close and to open. From these data it is apparent that the upstream depth must be about 2 feet before the gate starts to close and the difference in head may vary between 0.2 feet and 1.5 feet depending upon upstream and downstream levels. The data on the elevations at which the gate started to open were obtained by stopping all inflow to the chamber upstream of the gate. Since this condition is not likely to occur in the field, these data are considered to have little significance in the design of the gate.

For design purposes similar charts should be obtained from the gate manufacturer for the various size gates to be used.

2.8.4 Sample Computation
Tipping Gate

Pertinent data

D = diameter
Q = quantity of flow
d = depth of flow
V = velocity
S = slope

Interceptor sewer
D = 36", Invert el. = 10.0
Water surface = 12.4

Combined sewer - Design Q = 100 cfs
D = 54", Invert el. = 16.0, S = 0.0026
Manning n = 0.013, V = 6.4 fps (full)

	Flow	Q cfs	d ft.	V fps	WS
DWF	= Dry-weather flow - av	0.5	0.3	1.8	16.3
	Dry-weather - peak	2.0	0.5	2.5	16.5
	1-year storm	60.0	2.5	6.7	18.5
	10-year storm	100.0	3.6	7.2	19.6

Distance from interceptor to regulator 100'

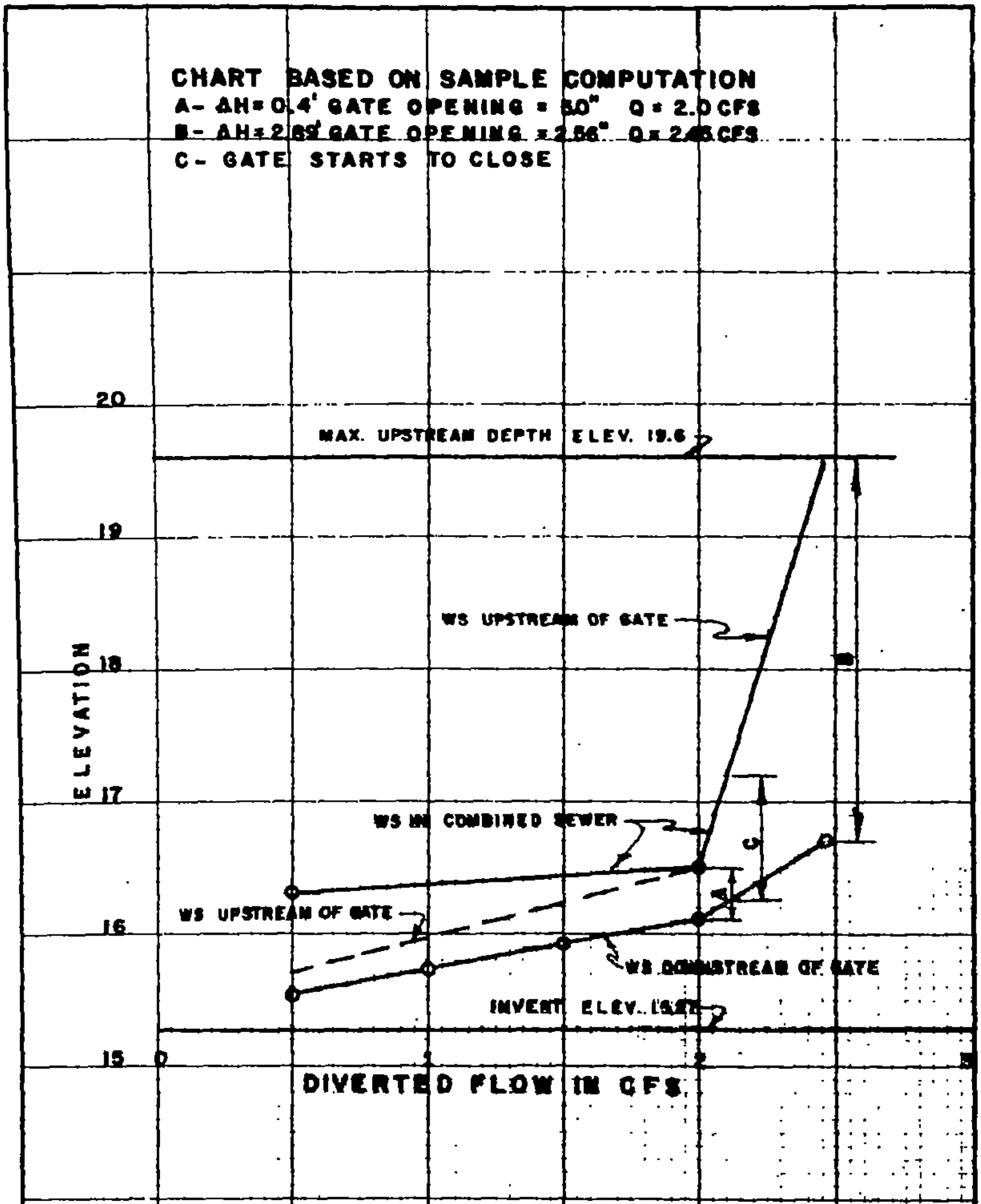
HGL = hydraulic grade line
EL = energy line

Design Q = 2 cfs

Branch Interceptor

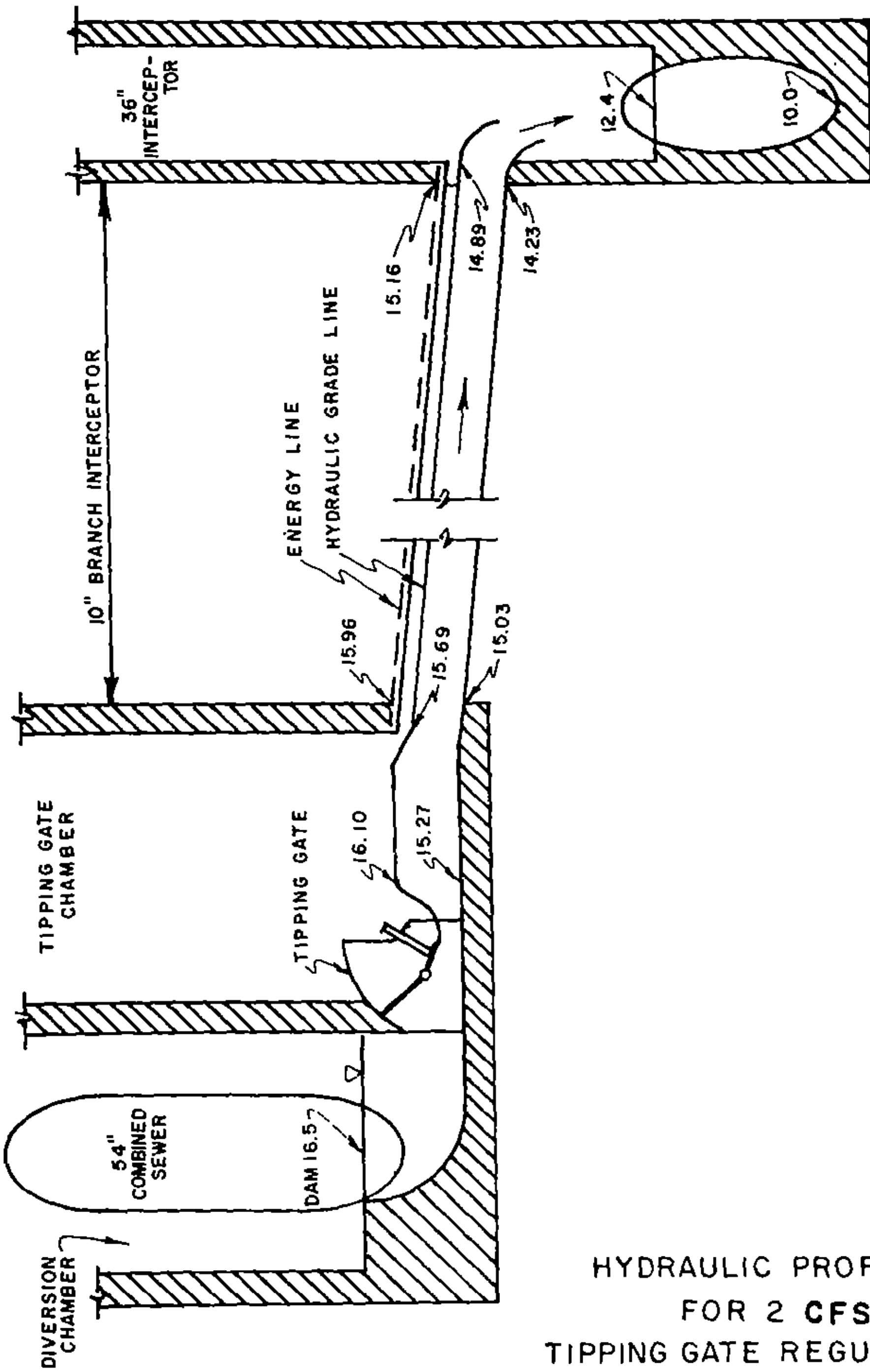
L = 100' n = 0.013 Dia. = 10"
s = 0.008 V = 3.7 fps (full–
V at 0.8 depth = 1.13 x 3.7 = 4.2 fps
$V^2/2g$ = 0.27

FIGURE 2.8.4.1



HYDRAULIC CONDITIONS FOR TIPPING GATE

FIGURE 2.8.4.2



HYDRAULIC PROFILE
FOR 2 CFS
TIPPING GATE REGULATOR

2.8.4 Tipping Gate

Diversion dam elevation

= 16.0 + 0.5 = 16.5

Try 12″ gate Figure 2.8.2

For 5.0″ opening and 10″ tail water differential head = 0.40′

Downstream WS = 16.5 – 0.4 = 16.1′

In Figure 2.1.3.1 for vertical orifice, downstream W.S. is 15.92. Therefore lower branch interceptor by 16.12 – 16.10 = 0.02 from that shown in Figure 2.1.3.1.

	ELEVATION		
Branch Interceptor	Invert	HGL	EL
Downstream invert 14.25 – 0.02	14.23		
Neglect critical depth 14.23 + 0.8 (0.83) = 14.23 + .66		14.89	
$V^2/2g = 0.27$			15.16
Friction loss 100 x 0.008 = 0.80			
Upstream end	15.03	15.69	15.96
Entrance loss 0.5 $V^2/2g = 0.14$	15.03	15.83	16.10
Neglect friction and bend loss in chamber		16.10	16.10
Set chamber invert 10″ below WS 16.10 - 0.83	15.27		

differential head = 16.50 – 16.10 = 0.40

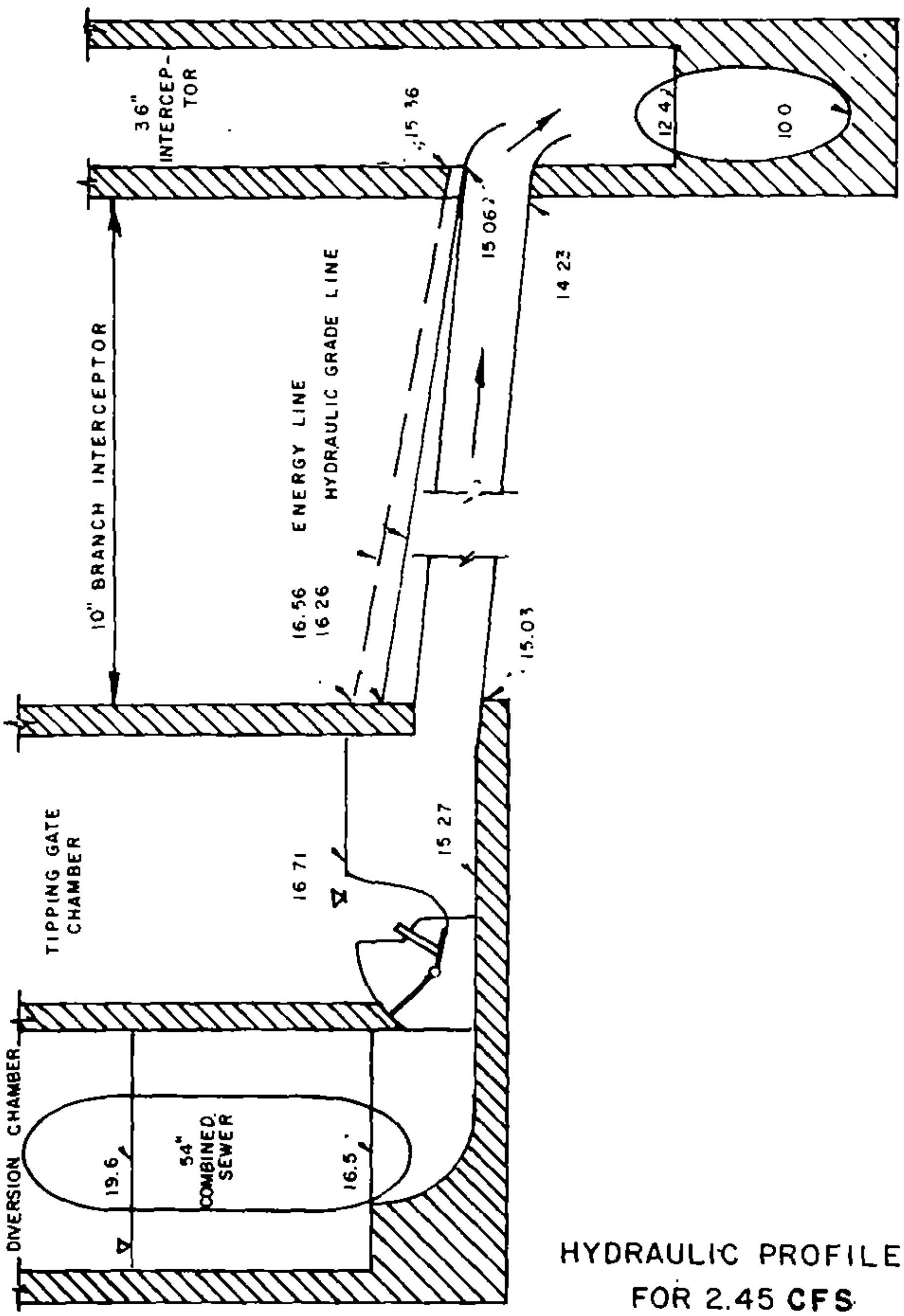
∴ Q = 2.0 cfs for 12″ gate and 5.0″ opening from Figure 2.8.2

Determine diverted flow in storm period

	10-year storm	1-year storm
HGL in combined sewer	19.60	18.50
Q diverted-cfs assume	2.4	2.2
10″ branch S =	0.012	0.010
Lower end top elev. 1/		

1/ This neglects critical depth at lower end and assumes HGL is at top of pipe.

FIGURE 2.8.4.3



HYDRAULIC PROFILE
FOR 2.45 CFS
TIPPING GATE REGULATOR

2.8.4 Tipping Gate

	10-year storm	1-year storm
Exit loss $V^2/2g$	0.30	0.25
Friction loss 100 x s	1.20	1.00
Entrance loss 0.5 $V^2/2g$	0.15	0.12
HG downstream of gate	16.71	16.43
differential head on gate	2.89	2.07
12" gate 2.56" opening Q =	2.5	2.1
Diverted Q = WWF	2.45	2.15
Ratio $\frac{WWF}{DWF} = \frac{WWF}{0.5}$	4.9	4.3

2.9 Cylindrical Gates

2.9.1 Description.

An isometric diagram of the cylindrical gate is shown in Fig. 2.9.1. Combined sewer flow is diverted by a dam through an opening in the side of the sewer into the gate chamber. The diverted flow drops through the horizontal orifice to the interceptor.

The operation of this device, when controlled by the sewage level in the branch interceptor, is shown in Figures 2.9.1B and 2.9.1C. When the level in the interceptor is low, as in Fig. 2.9.1B, the air-vent pipe prevents the formation of a vacuum in the interior of the gate and the gate stays open. When the level in the interceptor rises to a predetermined elevation, the sewage blocks the air-vent pipe, as shown in Fig. 2.9.1C. The entrainment of air produces a vacuum in the interior of the cylindrical gate and atmospheric pressure forces the gate down and closes the orifice.

The control of the gate by the sewage level in the combined sewer is shown in Figures 2.9.1D and 2.9.1E. When the level in the sewer is low the counterweight keeps the gate open, as in Fig. 2.9.1D. When the sewage level rises, the weight of the liquid on the conical part causes the gate to lower and close the orifice

DYNAMIC REGULATORS–AUTOMATIC

2.10 Cylinder-Operated Gates

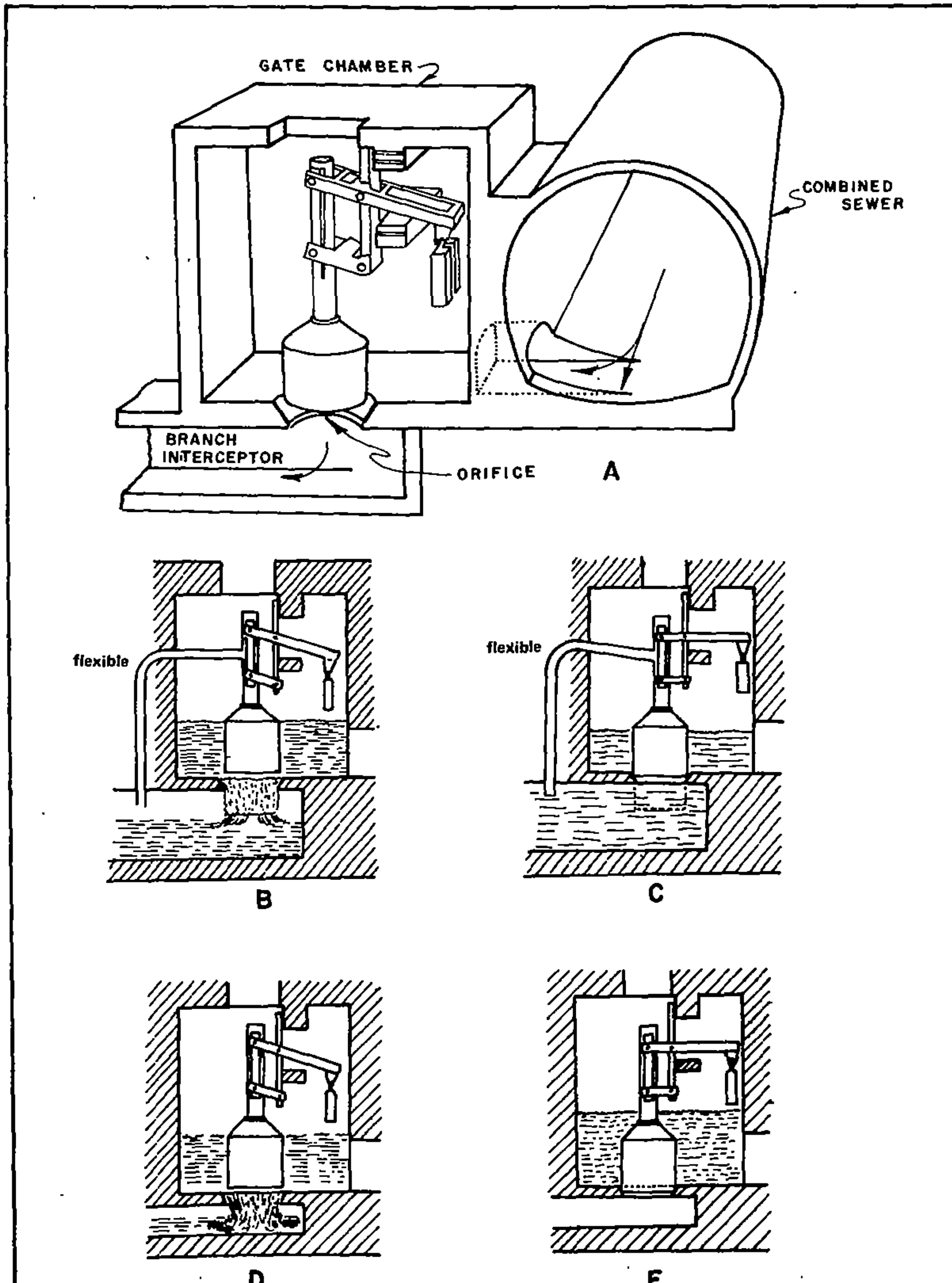
2.10.1 Description

The cylinder-operated gate may consist of two to four chambers: (1) A diversion chamber; (2) a regulator chamber containing sluice gate, cylinder and float or bubbler tube; (3) an equipment chamber when electrical equipment is required; and (4) a tide gate chamber, if required. On some deep and large chambers the diversion and flap gate chambers may be combined, but the other chambers should remain separate. Whenever possible it is desirable to construct the equipment chamber above ground.

The diversion chamber contains an overflow dam to divert the DWF into the adjacent regulator chamber. The top of the diversion dam is usually set to minimize the raising of the flowline upstream of the regulator during storm flows and prevent back flooding. The diversion channel invert is establised so that the peak DWF will be diverted without overtopping the dam. During wet-weather periods the excess flow goes over the dam to the tide gate chamber and thence to receiving waters. The opening between the diversion chamber and tide gate chamber is equipped with one or more tide gates.

The regulator chamber provides for a cylinder-operated sluice gate which governs the amount of flow to the interceptor. The action of the cylinder is related to the sewage level in the sewer by a sensing device which can be sensitive to either upstream or downstream flow conditions. The latter location is used if the main object of the regulator is to avoid overloading the interceptor and treatment plant. Generally, the sensing device is a float or a bubbler-tube. The cylinder is operated either by water, air or oil pressure. Floats may be used in conjunction with cylinders operated by water pressure to avoid the addition of compressed air equipment. During dry-weather periods the sluice gate is fully open. In wet-weather periods the rising sewage level will raise the float or increase the pressure in the bubbler tube so that the gate will partially close. The float or bubbler tube is located in

FIGURE 2.9.I



APPLICATIONS OF CYLINDRICAL GATE

Courtesy Neyrpic Canada Ltd.

a special well connected to the flow channel by a telltale passage.

When the sensing device is located downstream of the gate it is generally necessary to install a control device to maintain subcritical flow in the regulator chamber. One control which is satisfactory is vertical timber stop logs used to decrease the channel width. A vertical slide gate also can be used as a control to act either as a weir or an orifice on the bottom of the channel. The use of vertical wood stop logs has the advantage that the width of channel opening can be adjusted in the field, and there is nothing to impede the discharge of debris which may be carried on the bottom of the channel or may be floating on the surface of the flow.

While the hydraulics of the regulator can be computed, adjustments are usually required in the field to accommodate actual flow conditions. Usually the float or bubbler tube is set to act when the flow level is about one inch above the actual peak dry-weather flow line to insure that all sanitary flow in dry weather is diverted to the interceptor.

Water used as a pressure medium is usually obtained from the public water supply. Prevention of cross-connection hazards will require the use of check valves, vacuum breakers, and air breaks in drain lines, based on effective design criteria. Since the pressure in the water system may vary, the hydraulic cylinder is usually designed to operate on a minimum pressure of 25 psi. For small gates this pressure is adequate but for large gates a very large hydraulic cylinder would be required. None of the hydraulic cylinder manufacturers will guarantee cylinders for water operation. Therefore, the gate manufacturer is required either to build the cylinder or go to a speciality manufacturer for it. The chief advantage of the use of water is that no electrical power is required and, hence, the regualtor functions during power failures, and a spearate chamber is not needed for installation of air compressors or electrical equipment. Tightening of the packings around the piston rod and tail rod, if overdone, may increase the friction forces. Sometimes valves become inoperative due to rust or scale in the water supply. To prevent this a strainer should be installed in the supply line. Maintenance checks are necessary to insure that clogging of the strainer does not cause gate malfunctions.

When air is used as a medium for operating the gate a separate chamber is provided for the air compressors and electrical equipment. Air has been used at pressures of 90 to 100 psi. The disadvantages of this system are: (1) Electrical power is required which is subject to failure; (2) a separate chamber must be provided to house the electrical and compressed air equipment; and (3) difficulty is experienced in maintaining electrical equipment in subsurface chambers. Some jurisdictions using air pressure for cylinder operation have converted to oil pressure.

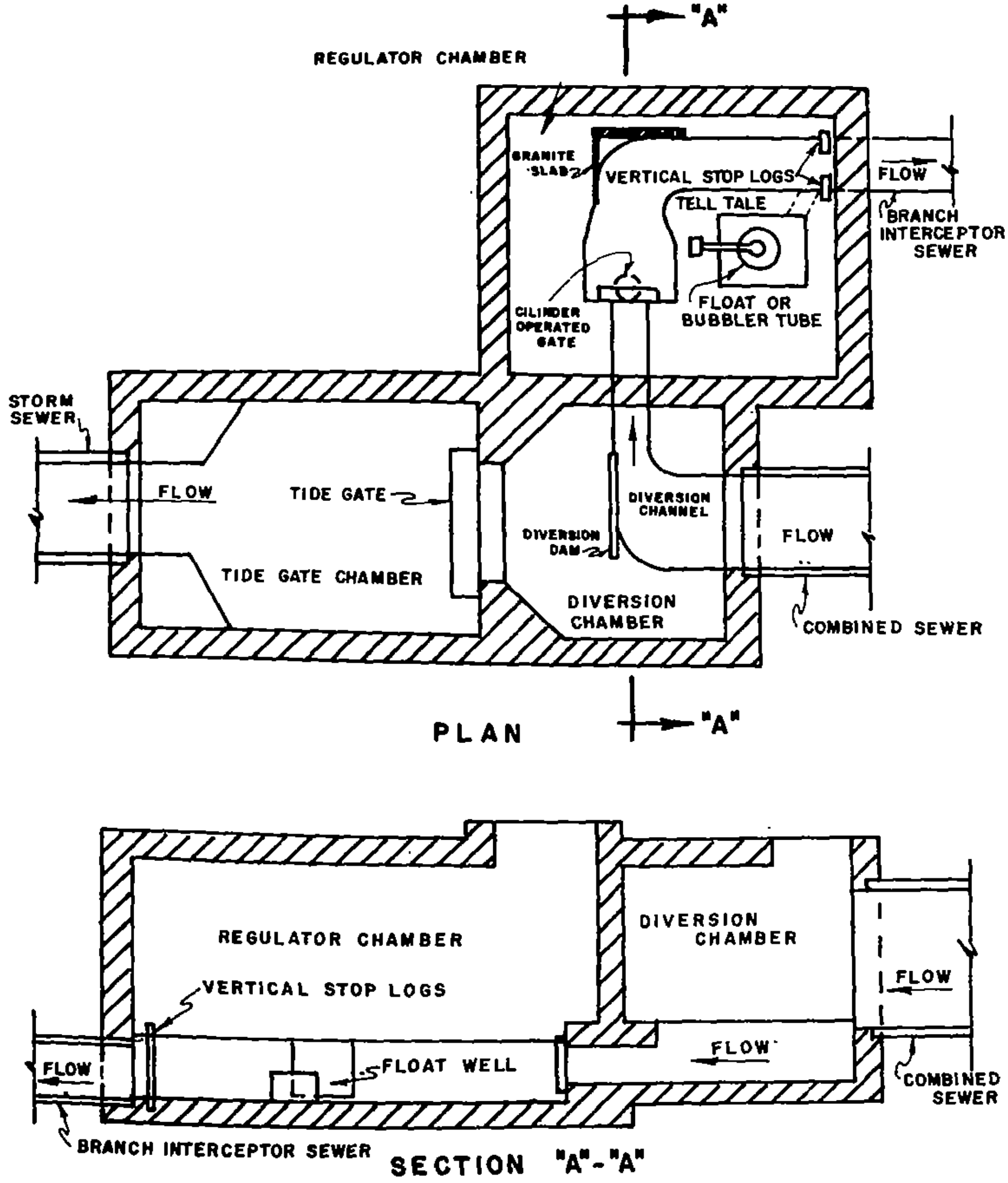
Recent practice in cylinder-operated gates seems to favor oil as a medium rather than air or water. Oil has been used at pressures of 350, 750, and 3000 psi but pressures from 600 to 750 psi are favored. To reduce corrosion, a separate chamber, preferably located above ground, is provided for electrical and pumping equipment. The use of oil results in less corrosion of valves and cylinders than the use of air or water. Smaller cylinders are needed to operate the gate due to the higher pressures used. The disadvantages are similar to those listed for air cylinders.

A typical plan of a cylinder-operated sluice gate regulator using water pressure is shown on Fig. 2.10.1.1. A schematic diagram of a cylinder-operated sluice gate regulator using water pressure is shown in Fig. 2.10.1.2 and one using oil pressure in Fig. 2.10.1.3.

The water-pressure operated cylinder, as shown in Fig. 2.10.1.2, functions basically as follows: (1) Water supply is usually obtained from a public water supply system which should be protected with adequate backflow prevention devices; (2) cylinders should be sized and designed for actual pressure of at least 5 psi less than minimum available pressure specified by the user; and (3) water pressure on the cylinder is controlled by a 4-way valve of the vertical plunger type which is actuated by a float in a well connected to the channel downstream of the sluice gate.

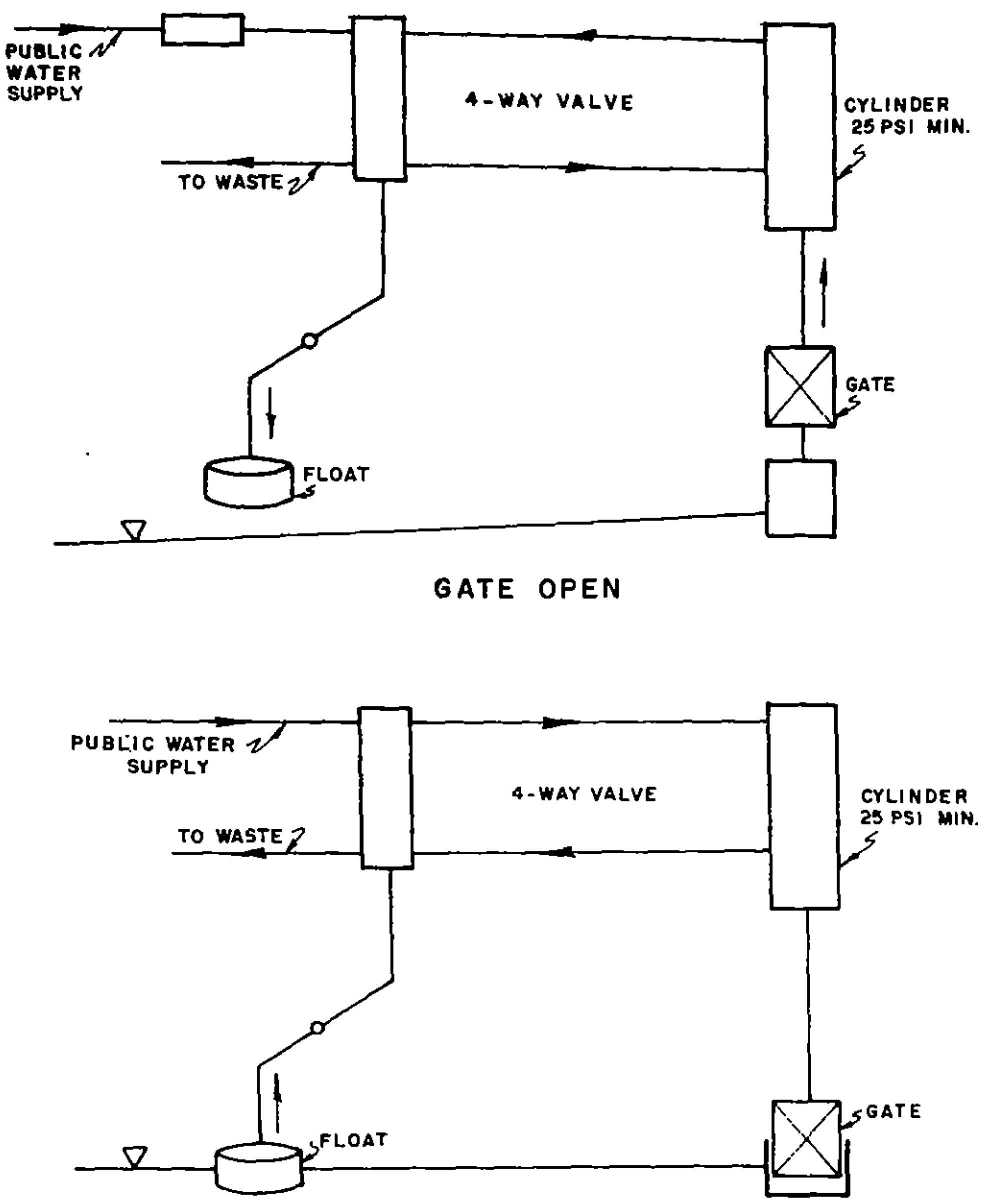
There are four pipe connections to the 4-way valve: (1) From the water supply; (2) to waste; (3) to the top of the hydraulic cylinder; and (4) to the bottom of the hydraulic cylinder. In dry-weather periods the float is down, the valve is up and the water pressure is supplied to the bottom of the hydraulic cylinder to keep the sluice gate wide open. During storm periods when the flow line in the channel reaches a predetermined level, the float rises, causing the 4-way valve to lower. This results in admission of water pressure in the bottom of the cylinder, thus causing the sluice gate to close. As the discharge through the orifice decreases the flow line in the regulator chamber falls, causing a reversal of this procedure. Thus the gate will "hunt" for its proper position. A needle valve is used to control the

FIGURE 2.10.1.1



CYLINDER-OPERATED GATE

FIGURE 2.10.1.2

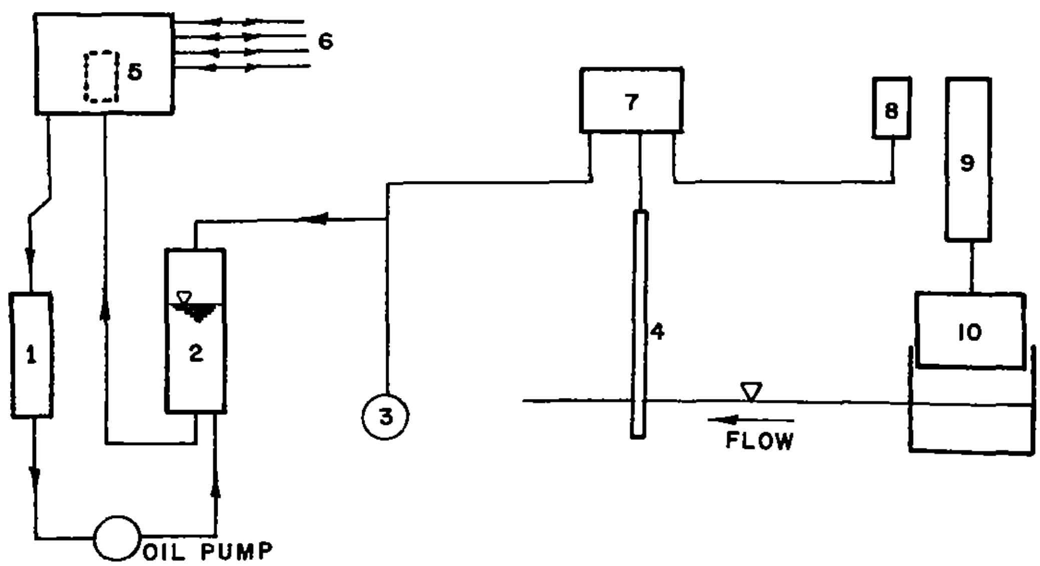


SCHEMATIC DIAGRAM

CYLINDER-OPERATED GATE
(WATER ACTUATED)

FIGURE 2.10.1.3

SCHEMATIC DIAGRAM



1 OIL RESERVOIR
2 ACCUMULATOR-OIL UNDER CONSTANT AIR PRESSURE
3 AIR COMPRESSOR
4 BUBLER TUBE
5 FOUR-WAY VALVE
6 OIL LINES TO POSITIONER & CYLINDER
7 ELECTRIC CONTROL
8 POSITIONER
9 HYDRAULIC CYLINDER
10 GATE

CYLINDER-OPERATED GATE (OIL ACTUATED)

rate at which the water pressure is transmitted to the cylinder to prevent rapid up-and-down movements of the gate. The usual installation will involve a vertical movement of the float of three inches and of the 4-way valve of.3/4 inch.

A cylinder-operated gate using oil pressure, as shown in Fig. 2.10.1.3, functions as follows: An air compressor supplies air continuously to a bubbler sensing device located downstream of the gate. The pneumatic pressure on the bubbler-tube is transmitted to a "positioner" on the cylinder. Oil is fed by an electric-motor-driven pump to the accumulator where the oil is kept under pressure by air supplied from the air compressor. When the position of the gate is out of balance with the level indicated by the bubbler-tube a signal from the positioner actuates a pneumatically operated four-way valve in the hydraulic control, which directs the oil to the top or bottom of the cylinder as required to open or close the gate.

Regulators of this type have been installed without a "positioner." Its use results in less "hunting" by the gate and is recommended.

A manual or diesel operated pump should be provided for gate operation in case of power failure.

The major advantage of this regulator is that the gate position can be transmitted to a remote control point either by a pneumatic signal, for distances up to 800 feet, or by an electrical signal to any distance. Such a system can also permit positioning of the gate from the remote control point.

2.10.2 Design Guidelines.

Obtain all pertinent data for the combined sewer at the proposed location of the regulator. This will include diameter, invert elevation, slope, average and peak dry-weather flow and peak storm flow. Obtain similar data for the interceptor at the proposed location of its junction with the branch interceptor. Also obtain data on high water levels in the receiving waters. If the interceptor is being designed in conjunction with the regulator, assume an elevation for the interceptor and adjust as found necessary by subsequent computations for the regulator.

After making a preliminary layout the energy and hydraulic grade lines should be computed for the branch interceptor, regulator and combined sewer. Insofar as possible, the designer should select elevations that will result in uniform flow. If non-uniform flow occurs it may be necessary to compute backwater or drawdown curves.

Initial design should be made on the basis of diverting the maximum dry-weather or peak sanitary flow, when peak sanitary flow occurs in both the interceptor and branch interceptor. The elevation of the water surface in the combined sewer as thus computed should be below the elevation of the diversion dam, which usually is established as six inches above the invert of the combined sewer. The sizes and elevations of the proposed structures should be adjusted as necessary so that this computed water surface is just below the dam elevation.

The hydraulic and energy grade lines also should be determined for the maximum storm flow and peak sanitary flow in the combined sewer and peak sanitary flow in the interceptor. If the receiving waters into which the combined sewer discharges are subject to variation in elevation the effect of this variation should be included in the computations.

Good design requires that the water surface in the float well will be dependent on the flow through the sluice gate and not be affected by flow conditions downstream in the branch interceptor or interceptor, and the diversion chamber and tide gate, if used, will not raise the water surface in the combined sewer during storm flows to an extent that damage from flooding will occur upstream of the regulator. If there are downstream constraints on the interceptor and if excessive flows are diverted to the interceptor, it may be possible for the hydraulic gradient to rise sufficiently in the regulator chamber to cause the float to close the sluice gate completely, thus eliminating all flow interception.

2.10.3 Design Formulas

The sample computations given are based on the hydraulic formulas outlined below. The designer may wish to analyze the problem in more detail than is given, by reference to texts on hydraulics.

As a result of turbulent flow in the regulator during periods of peak flow and because of clogging, it is doubtful if any regulator will act exactly as designed. The designer must use judgment as to how precise his computations shall be made. The following are design considerations:

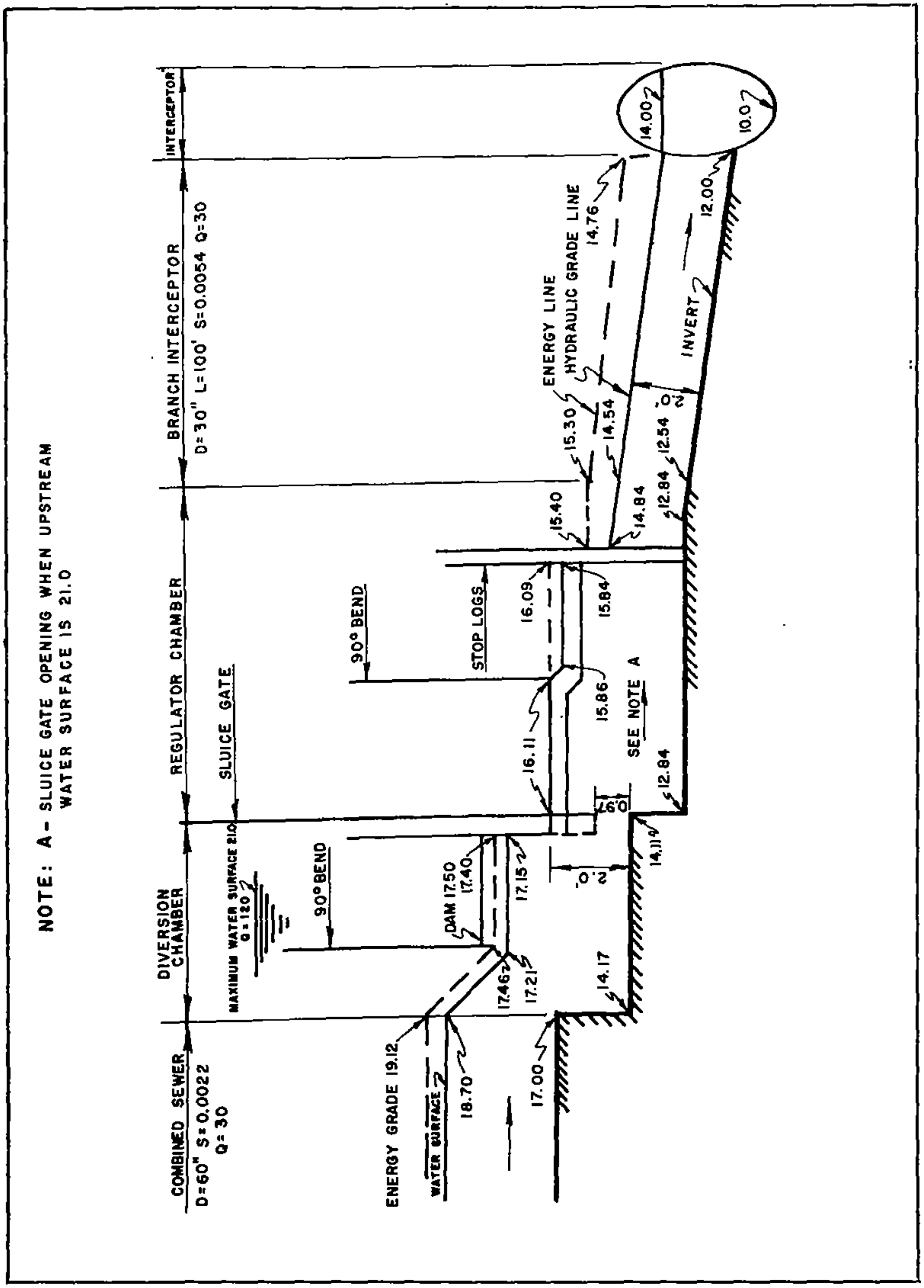
a. Friction losses in pipes and channels. Friction losses in the computations herein are based on the Manning formula using a constant *n* friction coefficient with variation in depth.
b. Contraction or inlet loss.
   $H = K(V_2^2/2g - V_1^2/2g)$

where H = head loss in feet

$K = 0.5$ for sharp-cornered entrance
$K = 0.1$ for gradual transition
V_1 = upstream velocity in fps
V_2 = downstream velocity in fps

c. Enlargement or outlet loss

FIGURE 2.10.4



HYDRAULIC PROFILE FOR REGULATOR
WITH CYLINDER-OPERATED GATE, Q OF 30 CFS

$H = K(V_1{}^2/2g - V_2{}^2/2g)$

where H = head loss in feet

K = 1.0 for sudden enlargement

K = 0.2 for gradual transition

V_1 = upstream velocity in fps

V_2 = downstream velocity in fps

d. Loss in 90-degree bend

$H = 0.25 - V^2/2g$

where H = head loss in feet

V = velocity in fps

e. Flow through orifice

$Q = CA\,(2gH)^{1/2}$

where H = head loss in feet

Q = discharge in cfs

A = area of orifice in square feet

C = 0.7 for sluice gate or vertical orifice

C = 0.6 for horizontal plate orifice

f. Discharge through opening between vertical stop planks

$Q = 3.09\,bH^{3/2}$

where Q = discharge in cfs

b = opening width in feet

H = total head upstream of stop planks

The above is applicable only if the downstream water depth is 2/3 or less of the upstream head.

g. Critical depth.

Critical depth may be determined for rectangular and circular conduits from Fig. 26 and for circular conduits from Table XVI both of which are in ASCE Manual No. 27.

h. Backwater and drawdown curves.

these curves may be computed by either of the methods illustrated in Tables XV and XVI in ASCE Manual No. 27.

2.10.4 Sample Computation
Cylinder-Operated Gate

Pertinent data

Interceptor sewer

D = 60", Invert = el. 10.0, W.S. = el. 14.0

Combined sewer

D = 60", Invert = El. 17.0, S = 0.0022

Manning n = 0.013 V (full) = 6.2 fps

Q	cfs	d ft.	W.S.	V fps
DWF = Av. dry-weather	15	1.20	18.20	4.3
Peak dry-weather	30	1.70	18.70	5.2
Peak storm [1]	120	4.00	21.00	7.0

[1] Includes peak dry-weather flow

Distance from interceptor to regulator on combined sewer is 100 ft.

For hydraulic profile see Figure 2.10.4

HGL = hydraulic grade line

EL = energy line

D = diameter, V = velocity, Q = discharge

d = depth of flow

S = slope

L = length

2.10.4 Cylinder-Operated Gate

	ELEVATION		
Interceptor – Peak dry-weather flow	Invert	HGL	EL
Branch Interceptor n = 0.013	10.00	14.00	
D = 30 in., Q = 30 cfs, s = 0.0054			
d = 0.8 x 2.5 = 2.0 ft.			
V = 1.13 x 6.2 fps			
$V^2/2g$ = 0.76 ft.			
Lower end	12.00	14.00	
Pipe outlet loss 1 (0.76 – 0) = 0.76			14.76
Upper end			
Friction loss 100 x 0.0054 = 0.54	12.54	14.54	15.30

Regulator Chamber

Say b = 2.5 ft., d = 2.0 ft.	12.54	14.54	15.30
$V = \frac{30}{2.5 \times 2.0} = 6.0$ fps			
$V^2/2g$ = 0.56			
Pipe inlet loss = 0.5 (0.76 – 0.56) = 0.10			15.40
HGL = 15.40 – 0.56		14.84	
Invert = 14.84 – 2.0	12.84		

Determine if flow is stable from Figure 26 ASCE Manual 37
Q = 30 cfs   b = 2.5 feet
dc/b = 0.65
dc = 0.65 x 2.5 = 1.62
$\frac{2.0}{1.62} = 1.24 > 1.15$ Flow is stable

However to reduce chance of backwater effect from interceptor raise flow line 1.0 foot by inserting vertical stop logs in channel

d = 3.0 feet	12.84	15.84	
$V = \frac{3.0}{2.5 \times 3.0} = 4.0$ fps			
$V^2/2g$ = 0.25 ft.			16.09

2.10.4 Cylinder-Operated Gate

	ELEVATION		
	Invert	HGL	EL
Determine stop log opening			
$Q = 3.09\ b\ H^{3/2}$			
$30 = 3.09 \times b \times (3.0 + 0.25)^{3/2}$			
b= 1.65 feet (opening)			
Condition upstream stop logs	12.84	15.84	16.09
Channel friction loss			
L = 10 ft. n = 0.015			
V = 4.0 fps r = 0.88			
From nomograph S = 0.0018			
Friction loss = 10 x S = 0.02	12.84	15.86	16.11
Assume complete loss of velocity of discharge from sluice gate from impact on granite slab:			
Upper end of channel	12.84	16.11	16.11

Regulator Chamber

Head loss in sluice gate
Try 30 in. x 24 in.

$Q = CA\sqrt{2gH}$
$30 = (0.7)\ (2.5)\ (2.0)\ (8.03)\sqrt{H}$
$H = 1.04$ ft.

Diversion Chamber

	Invert	HGL	EL
Invert = 16.11 – 2.0	14.11		
16.11 + 1.04		17.15	
Approach Channel			
b = 2.5 ft.			
d = 17.15 – 14.11 = 3.04 ft.			
$V = \dfrac{30}{2.5 \times 3.0} = 4.0$ fps			
$V^2/2g = 0.25$			17.40
Neglect friction loss in channel			
Bend loss 0.25 (0.25) = 0.06	14.17	17.21	17.46
Required upstream of bend		17.21	17.46
Available in combined sewer	17.00	18.70	19.12

Dam elevation 17.00 + 0.5 = 17.50
\> 17.21 Design OK

2.10.4 Cylinder-Operated Gate

Determine sluice gate opening during peak storm

$$Q = 120 \text{ cfs in combined sewer}$$
$$HGL = 21.00 \text{ in combined sewer}$$
$$H = 21.00 - 16.11 = 4.89 \text{ ft.}$$
$$Q = CA\sqrt{2gH}$$
$$Q = 30 \text{ cfs through sluice gate}$$
$$30 = (0.7)\,(2.5)\,(d)\,(8.03)\sqrt{4.89}$$
$$d = 0.97 \text{ ft. — height of gate opening}$$

Thus gate travel is 2.0 – 0.97 = 1.03 ft.

The following adjustments could be made in the foregoing design. Since the computed HGL in the diversion chamber is 0.29 feet (17.50 - 17.21) below the top of the dam the invert of the structure between the diversion chamber and the stop logs could be raised 0.29 feet. Since the stop logs control flow upstream thereof, the HGL downstream of the stop logs could be lowered by decreasing the length and/or increasing the diameter of the branch interceptor. Since the drop in EL at the stop logs is 0.69 feet (16.09 - 15.40) any rise in water surface of the interceptor exceeding 0.69 feet will result in backwater effect on the regulator.

2.11 Motor-Operated Gates

2.11.1 Description.

A regulator using a motor-operated gate consists of two to four chambers; (1) A diversion chamber; (2) regulator chamber containing the sluice gate and a sensor; (3) a motor chamber; and (4) a tide gate chamber, when required. On some deep and large chambers the diversion and tide gate chambers may be combined but the other chambers should remain separate. Whenever possible it is desirable to construct the motor chamber above ground. The diversion chamber is as described for cylinder-operated gates in 2.10 of this Section.

The regulator chamber contains a motor-operated sluice gate which governs the amount of flow diverted to the interceptor. The action of the gate relates to the sewage level by a sensing device which can be used to respond to flows either upstream or downstream of the sluice gate. The latter location is used if the main object of the regulator is to avoid overloading the interceptor and treatment plant. The sensor could be a sealed electrode type, pressure cell type or possibly pressure-sensitive electric type. A float or compressed-air bubbler tube could also be used. The design of the regulator chamber is similar to that of the cylinder-operated gate (see 2.10.)

2.11.2 Design Guidelines.

Guidelines for hydraulic design are the same as those for the cylinder-operated gate. Design formulas for sample computations for this regulator are as indicated for the cylinder-operated gates.

2.12 External Self-Priming Siphon

2.12.1. Description

This device is currently in use in Europe and is a new flow control method which could lend itself to "total systems control."

The external self-priming siphon uses an exterior device to prime the siphon and the design of the siphon conduit is not as critical. A siphon of this type, as shown in Fig. 2.12.1, was proposed by A. Moan and Y. Ponsar of France, for use as a flood-stage or tide-water check valve (tide gate) to protect the interceptor. When the upstream water surface is higher than the downstream water surface the sewage will flow through an orifice plate in the top of the priming tube and through the priming tube to the downstream side. This flow will carry air with it, causing a partial vacuum on the underside of the orifice. By connecting the summit of the siphon to the priming tube, a vacuum is created on the siphon summit, causing the siphon to discharge. To prevent siphonage in the opposite direction a float and air valve device is provided so that when the downstream level raises the float an air valve is open, allowing air at atmospheric pressure to enter the summit. The use of this type siphon with a "Ponsar Regulator" as a combined sewer regulator is described in subsequent paragraphs.

The Ponsar siphon also has been used in waste water treatment plants but, so far as is known, has not been used in either the United States or Canada in connection with combined sewer regulators. The treatment plant in Geneva, Switzerland, utilizes this type of siphon to automatically distribute flow equally to the remaining number of treatment units in operation if one or more is taken out of service, and to divert flows in excess of fixed maximums to other units of the treatment plant.

Its use also has been suggested for diverting a predetermined amount of combined sewer flow to a plant, as shown in Fig. 2.12.1. The minimum flow that can be diverted by this type of siphon is governed by the fact that, to prevent clogging, the priming tube should have a minimum diameter of eight inches and the orifice a minimum diameter of five inches. Using these sizes the application of the Ponsar siphon to diversion flows less than 8 cfs would not seem practical.

The priming tube is used to develop negative pressures in the siphon summit. If the inlet and outlet branches of a siphon are submerged, a priming tube can be utilized effectively to establish siphonage, provided air evacuation velocities are developed in the priming tube and its cross-sectional area is large enough to pass suspended materials transported by the incoming waste water flow.

Upon establishment of appropriate vacuum conditions within the siphon, the water column will rise to the summit of the siphon and flow will be established over the siphon crest or weir into the discharge branch. By controlling the intensity of vacuum conditions within the siphon, the discharge can be effectively regulated to desired discharge limits.

Basic Components:

The siphon installation comprises three basic elements: (1) An upflow branch with a priming tube within the upflow branch; (2) a vacuum chamber with a vacuum regulating device; and (3) a discharge branch. A self-contained siphon installation incorporating these basic elements is shown in Fig. 2.12.1.

The priming tube consists of a priming pipe with an orifice plate attached at the top. The orifice is set at a level high enough above the crown of the inlet sewer to assure continuously complete submergence at entry to the siphon upflow branch. The lower end of the priming pipe extends to the flow level of the receiving conduit where it must be submerged below the lowest downstream discharge level.

The orifice serves to establish a jet discharge which draws entrapped air from the space directly under the orifice, surrounding the jet contraction, and from the siphon summit through an air suction pipe installed for that purpose. The air and waste water mixture is drawn down the priming pipe where the air escapes to the downstream water surface. The rate of air evacuation tends to increase with the length of the priming pipe. The pipe connection from the siphon summit to the priming pipe, immediately below the orifice serves to remove air from the siphon initially and after full flow is established.

Consideration of Flow Conditions in Priming Pipe

Experimental work reported by Kalinske in "Hydraulics of Vertical Drains and Overflow Pipes, Bulletin No. 26, Studies in Engineering, University of Iowa," provides data on downdraft flow in pipes roughly comparable to flow conditions in a priming tube. In accordance with Kalinske's findings, the maximum ratio of air removal to water flow is approximately 0.65 when discharging about 1.1 cfs of water through a 6-inch diameter pipe 6.67 ft. long, and is approximately 1.0 when the water discharge is at a rate of 1.0 cfs in a priming tube 11.0 ft. long. For short pipes of 2- to 3- ft. length, jet flow occurs without touching the walls of the pipe. The negative pressure immediately below the pipe entrance increases with the discharge and length of pipe. Although longer pipes will produce a higher vacuum the discharge does not increase correspondingly. In general, the ratio of head "H" on the priming pipe to pipe diameter "D" is proportional to the Froude number, particularly when the priming pipe is not flowing full. However, the Reynolds number becomes important when the priming pipe begins to flow full. The critical values of H/D at which the latter occurs may be estimated at approximately 0.9 to 1.1 for pipe lengths ranging from 20 to 50 pipe diameters. The orifice may be expected to modify the critical head.

Air bubbles will tend to rise against the downward water flow at a velocity of approximately 0.75 fps. Therefore, downward current velocities greater than 0.75 fps must be maintained to evacuate air. For effective priming, a necessary condition is that the downflow rate of liquid in the priming pipe must exceed 1 fps and should preferably be about 2 fps. As air is removed from the siphon an equivalent volume of water will be drawn up into the siphon and into the priming tube. Similarly, after flow is established over the crest of the siphon, more air will